

MIL-HDBK-1025/6  
15 MAY 1988  
SUPERSEDING  
NAVFAC DM-25.6  
JULY 1981

# MILITARY HANDBOOK

GENERAL CRITERIA FOR  
WATERFRONT CONSTRUCTION



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ABSTRACT

This handbook defines the basic criteria to be used in the design of elements used in waterfront construction. This handbook is intended for experienced engineers. The contents cover general requirements concerning piling, deck and substructure framing, and hardware for waterfront structures, as well as specific criteria for various types of installations. It also includes a section on the strength evaluation of existing waterfront structures and a section on the deterioration of waterfront structures is also included.



## FOREWORD

This handbook has been developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command (NAVFACENGCOM), other Government agencies, and the private sector. This handbook was prepared using, to the maximum extent feasible, national professional society, association, and institute standards. Deviations from this criteria, in the planning, engineering, design, and construction of Naval shore facilities, cannot be made without prior approval of NAVFACENGCOMHQ Code 04.

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<u>Criteria Manual</u>	<u>Title</u>	PA
MIL-HDBK-1025/1	Piers and Wharves	LANTDIV
MIL-HDBK-1025/2	Dockside Utilities for Ship Service	LANTDIV
MIL-HDBK-1025/3	Cargo Handling Facilities	LANTDIV
DM-25.4	Seawalls, Bulkheads, and Quaywalls	LANTDIV
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NOTE: Design manuals, when revised, will be converted to military handbooks.

This handbook is issued to provide immediate guidance to the user. However, it may or may not conform to format requirements of MIL-HDBK-1006/3 and will be corrected on the next update.

## GENERAL CRITERIA FOR WATERFRONT CONSTRUCTION

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Section 1: INTRODUCTION

1.1 Scope. This handbook contains general criteria for the design of piling, deck, and substructure framing and bracing, and hardware and fittings for waterfront construction. Unless indicated otherwise, these criteria also apply to the design of offshore structures.

1.2 Cancellation. This handbook cancels and supersedes DM-25.6, General Criteria for Waterfront Construction, (July 1981).

## Section 2: PILING

2.1 General Requirements2.1.1 Capacity

2.1.1.1 Capacity as a Structural Member. For pile sections embedded in the ground refer to NAVFAC DM-7, Soils and Foundations Series. For sections freestanding in water, treat pile as a column having an unbraced length as shown in Figure 1. Where, due to long-term creep effects, the use of the coefficient of subgrade reaction would be inappropriate or if one is unavailable, or for conditions not covered by Figure 1, the following assumptions may be made:

a) In soft, cohesive soils, the point of fixity may be assumed to occur at a depth of 10 ft (3.05 m) below the mudline for piles having modulus of elasticity to moment of inertia ratio (EI) of  $10 \times 10^9 \text{ lb-in}^2$  (0.453 kg) or less. The point of fixity may be assumed to occur at a depth of 12 ft (3.66 m) below the mudline for piles having an EI greater than  $10 \times 10^9 \text{ lb-in}^2$ . E equals Modulus of Elasticity of Pile in pounds per in.<sup>2</sup> and I equals moment of inertia of pile in in.<sup>4</sup>.

b) In loose, granular soils and in medium cohesive soils, the point of fixity may be assumed to occur at a depth of 8 ft (2.44 m) below the mudline for piles having an EI of  $10 \times 10^9 \text{ lb-in}^2$  or less, and at a depth of 10 ft below the mudline for piles having an EI greater than  $10 \times 10^9 \text{ lb-in}^2$ .

c) For other cases, assume a point of fixity at a depth of 5 ft (1.5 m) below the mudline.

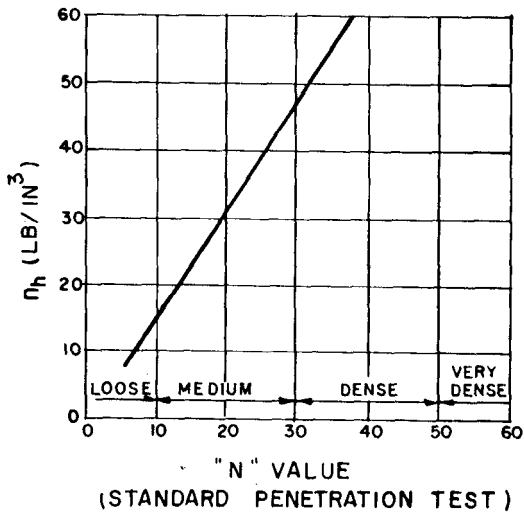
The effective length factor K (see Figure 1) shall be taken as:

a) 0.75 - when the deck structure is light, the piles have minimum embedment into the pile cap and there is no provision for moment transfer into the deck structure.

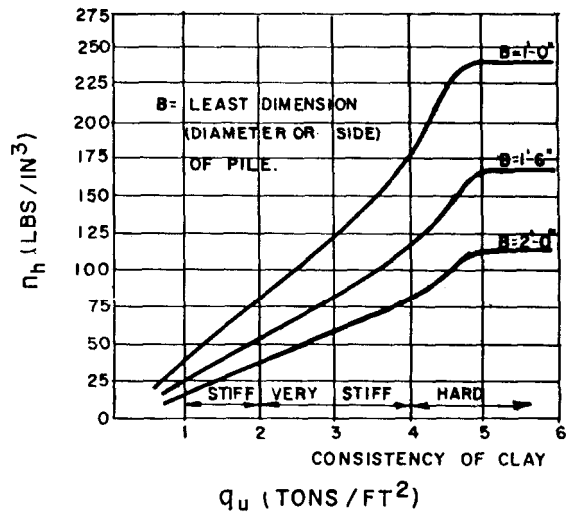
b) 0.67 - when the deck structure is light and a provision is made for moment transfer by embedment or other device into the deck structure.

c) 0.50 - when the deck structure is heavy and a positive means for moment transfer is provided.

NOTE : These provisions do not apply if embedment is less than 10 ft into firm material or 20 ft (6.10 m) into soft or loose material. If lesser penetration is provided, assume that the piles are hinged at their lower end. The indicated effective length factors apply if batter piles (minimum batter of one horizontal to three vertical) are provided to resist full lateral loads, i.e., the plumb piles are not intended to resist lateral loads. If no batter piles are provided, increased K factors shall apply corresponding to condition where sidesway can occur.



USE THIS CHART TO APPROXIMATE  $n_h$  FOR MEDIUM TO DENSE INORGANIC SILT AND SAND SOILS.



USE THIS CHART TO APPROXIMATE  $n_h$  FOR STIFF TO HARD CLAY SOILS.

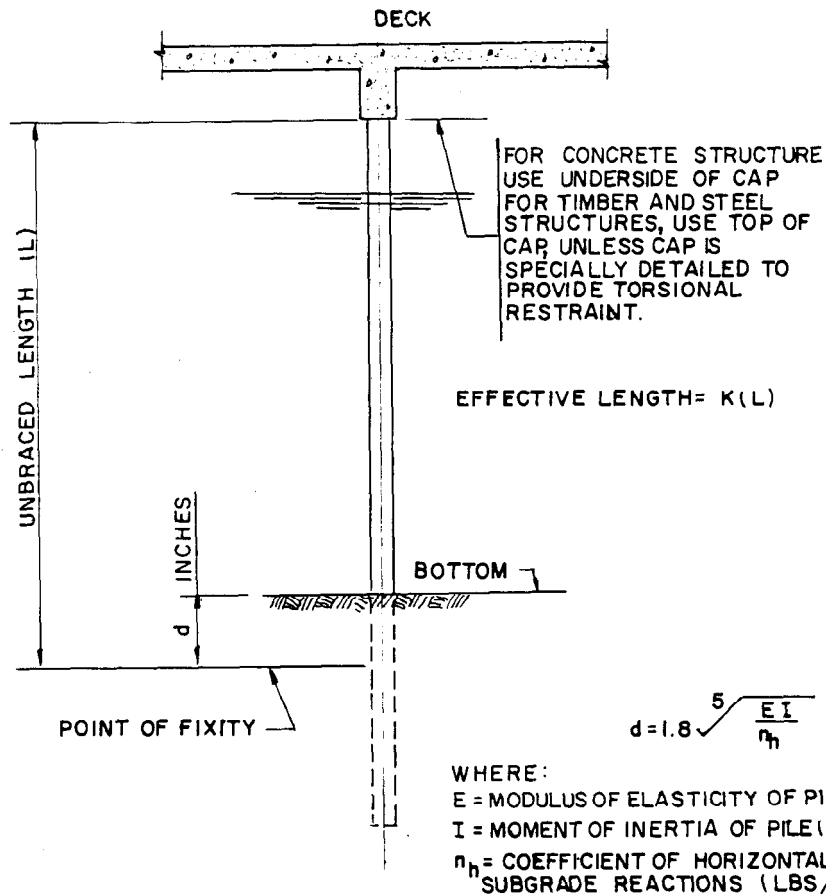


Figure 1  
Determination of Unbraced Pile Length

Piles shall be designed for a minimum eccentricity of 0.10 times the equivalent diameter of the pile. The moment resulting from this minimum eccentricity is not additive to the moments indicated by analysis of the applied loads.

2.1.1.2 Capacity of the Ground to Support the Pile. Refer to provisions of the NAVFAC DM-7 series, pertaining to friction, end bearing resistance, and settlements of single piles and pile groups.

2.1.1.3 Lateral Load Capacity. For piles spaced more than three diameters center-to-center, assume that the soil reacts laterally on an equivalent pile having a diameter equal to three times the actual diameter of the pile. For closer spacing, reduce the assumed equivalent diameter proportional to the spacing.

NOTE : A number of published design curves already include the effects of this spread of the load.

2.1.1.4 Capacity of Existing Piles. Refer to Section 6 for information pertaining to the capacity of existing piles.

## 2.1.2 Details Applicable to All Pile Types

2.1.2.1 Minimum Penetration. The minimum penetration of piles shall conform to the following:

a) Penetrate sufficiently into an acceptable bearing stratum to distribute the pile load within the supporting capacity of the soil (refer to DM-7.01, Soil Mechanics, DM-7.02, Foundations and Earth Structures, and DM-7.03, Soil Dynamics. Deep Stabilization. and Special Geotechnical Construction).

b) Penetrate sufficiently below probable future dredge depth to distribute the pile load within the supporting capacity of the soil. Discount resistance of soil which reasonable expectation indicates may be removed.

c) Minimum values. Although minimum values shall apply, if the pile penetration is less than 10 ft (3.05 m) into firm material and less than 20 ft (6.10 m) into soft or loose material, special provisions such as hardened tips driven into the refusing stratum, drilled sockets, or drilled dowels shall be made to secure the tips of the pile against lateral displacement due to inevitable eccentricities, lateral forces, and being out of plumb. The required lateral resistance shall be at least 5 percent of the design axial load. The effective length factors shall be increased as described above.

2.1.2.2 Tolerances on Installation. For piles fully, or near fully, embedded in the ground, the provisions of NAVFAC Guide Specifications (TS Series) apply. The following provisions relate to piling installed in open platforms where the piles project several feet or more above the mudline:

a) Slope shall generally be +4 percent from plumb, or specified batter, as a reasonable compromise between the needs of the design and the practicality of installation.

b) In locating the pile head, there is no limit, provided the structure can tolerate the revised pile spacing. However, residual stresses in the piles due to forcing the pile into the pile cap shall be considered in evaluating the column capacity of the pile. No increase (or additional increase) in allowable stress should be applied to stress combinations which include these residual stresses.

NOTE : For effective unbraced length, divided by radius of gyration ( $Kl/r$ ) between about 40 and 100, these effects can be substantial (refer to para. 6.5). But for fully embedded piles,  $Kl/r$  commonly is less than 40 and locked-in stresses can be neglected. Caution shall be exercised when using driving frames because they prevent lateral movements of the head of the pile and mask the existence of locked-in stresses. Ample edge distances shall be provided so that the piles will fit into the cap without excessive force or restraint. Allow for tolerance in the location of the pile head of at least 1.5 percent of the exposed height.

2.1.2.3 Minimum Spacing. The minimum spacing requirements of piles are as follows :

a) Provide for adequate distribution of the load on a pile group to the supporting soil.

b) No minimum values are specified other than practical limitations to avoid piles interfering with or intersecting each other. One technique is to use a center-to-center spacing equal to 5 percent of the pile length.

2.1.2.4 Pile Caps in Contact With the Ground. The piles shall be designed to carry the entire superimposed load with no allowance made for the supporting value of the material between the piles.

2.1.2.5 Connection of Piles to Caps. The following requirements pertain to the connection of files to caps:

a) No Tension in Piles--Timber Caps. Tops of piles shall be secured to caps with spiral-drive drift bolts, metal straps, or scabs.

b) No Tension in Piles--Concrete Caps. Tops of piles shall have a minimum 4-in. embedment.

c) No Tension in Piles--Steel H-Piling in Concrete Caps. Refer to requirements in para. 2.2.6.3.

d) Tension in Piles--Timber Piles in Concrete Caps. Shoulder and embed butts to satisfy requirements of shear stress in the timber and diagonal tension stress in the concrete. Timber connectors are permitted as an alternate method of stress transfer from cap to piles.

e) Tension in Piles--Concrete Piles in Concrete Caps. Dowel into cap.

f) Tension in Piles--Steel Piles in Concrete Caps. Calculate bond resistance as  $0.02 f'_c$  for contact surfaces which are cast "in the dry" and as 10 psi (68.95 kPa) for contact surfaces which are cast below water (tremie).

g) Tension in Piles--Timber Piles With Timber Cap. Provide scabs and shear bolts or provide metal straps.

2.1.2.6 Batter Piles. Connections to adjacent piles in group shall be capable of developing the calculated tension, but not less than a tension equal to one-half the compression load less the dead weight in the pile.

2.1.2.7 Splicing. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pile during installation and subsequent thereto. Splices shall be of adequate strength to transmit the vertical and lateral loads (including tensions), and the moments exceeding the allowable stresses for such materials as established in the DM-2 Series for Service Classification B. Except for piles that can be visually inspected after driving, splices shall develop at least 50 percent of the capacity of the pile in bending or the moment and shear that would result from an assumed eccentricity of the pile load of 3 in. (76.2 mm), whichever is the greater requirement. No minimum strength requirement is specified for pile splices that can be inspected and reinforced after driving.

2.1.2.8 Mixed Types or Capacities of Piling and Multiple Types of Installation Equipment or Methods. Mixed types or capacities of piling and different types of installation equipment or methods are permitted, provided that analysis is made of the effects on the superstructure of differential elastic shortening and settlement. Consider conducting load tests to evaluate differential settlements and spring values for piles.

2.1.2.9 Slope of Batter Piles. Unless special provisions are made for the difficulties of installation and the effects of diminution of the hammer blow on the capacity, keep the slope of the batter piles to one horizontal to two vertical or steeper (preferably 1 horizontal to 2.5 vertical).

## 2.2 Requirements for Specific Type of Piles

### 2.2.1 Untreated Timber Piles

2.2.1.1 Piles. Piles shall conform to ASTM D25, Specification for Round Timber Piles.

2.2.1.2 Cutoff. Cut off piles at or below permanent ground water level. In areas having semidiurnal tides, cutoff shall be at or below a level equal to R/3 above Mean Low Water (MLW), where R is the tidal range. In areas having a diurnal tide, cutoff shall be at MLW.

2.2.1.3 Borers. Untreated piles shall not be used in locations where they will be exposed to borers, except that use of untreated fender piles will be permitted where experience demonstrates that such use is justified. In general, untreated piles should not be used where they will be freestanding in salt or brackish water.

2.2.1.4 Seasoning. Seasoning for untreated timber piles is not required.

2.2.1.5 Protection for Tops of Piling. Protection for the tops of piling of untreated timber is not required.

2.2.1.6 Hardware and Fittings. Refer to Section 4 for information on hardware and fittings.

2.2.1.7 Species. Any species of wood may be used that will provide the necessary structural capacity and that will withstand the driving stresses.

2.2.1.8 Peeling. Peeling of untreated timber piles is not required.

## 2.2.2 Treated Timber Piles

2.2.2.1 Piles. Piles shall conform to ASTM D25, Specification for Round Timber Piles.

2.2.2.2 Preservative Treatment. Treated marine piling shall bear the appropriate American Wood Preservers Bureau (AWPB) Quality Mark as follows: MP-1 (dual treatment) for use in areas of extreme borer hazard and in marine waters where Limnoria and Pholadidea attack may be expected, or where oil slicks may contribute to borer attack, and MP-2 for other conditions where pholad attack is not expected. MP-4 treatment (water-borne preservatives) may be considered. For specific requirements at particular locations, consult NAVFAC Engineering Command, Pacific (PACDIV), Atlantic (LANTDIV) or North Divisions (NORTHDIV), Applied Biology Office. Refer to para. 5.9 for properties of treated wood.

2.2.2.3 Seasoning. Seasoning of treated timber piles is required prior to treatment.

2.2.2.4 Species. The preferred species is southern pine or douglas fir. Use of other species may be made subject to NAVFAC approval. AWPB Quality Control Standards include a requirement that the species be southern pine or douglas fir. It is not normally necessary to specify species separately. In areas where treatable soft woods are scarce, and if a treated pile is required, consider the use of concrete piling.

2.2.2.5 Protection for Pile Tops. Preferably, cut ends shall be treated by puddling creosote. Puddling is accomplished by using a sheet metal ring to form a reservoir on top of the pile. The reservoir is filled with creosote oil and left to stand for 8 to 12 hours. Alternative protection methods include coating pile tops with pitch (with or without sheet metal covers).

NOTE : Use of sheet metal covers as end protection for fender piles is discouraged because the covers are easily torn by impact and become a personnel hazard. However, sheet metal covers for bearing piles under cross caps provide good protection. In general, piling to be covered with other structural members shall be fitted with waterproof caps.

## 2.2.3 Untreated and Treated Timber Piles

2.2.3.1 Limitations on Use. Timber piles installed to end-bearing on rock, hardpan, caliche, or other semi-cemented materials require special care in installation to prevent damage.

2.2.3.2 Lagged Piles. Double lagging shall be connected to the basic pile material to transfer the full pile load from the basic pile material to the

lagging. The connection for single lagging shall be proportioned for one-half the pile load.

#### 2.2.4 Precast (Including Prestressed) Concrete Piles

2.2.4.1 Minimum Dimensions. The minimum dimensions are 12 in. (304.8 mm) for piles of uniform section and 8 in. (203.2 mm) for tapered piles.

2.2.4.2 Cover. The minimum clear cover for reinforcement for permanent installations in salt water shall be 3 in. (76.2 mm). For temporary installations and in fresh water, cover requirements shall conform to the requirements of NAVFAC DM-2.04, Concrete Structures, for normal exposure conditions.

2.2.4.3 Minimum Reinforcement. Excluding prestressed piles, the minimum longitudinal reinforcement shall be 1.5 percent of the total cross section.

2.2.4.4 Ties. Spirals or ties shall be provided for longitudinal reinforcement. Proportion spirals and ties in accordance with the ACI provisions for structural columns except provide additional ties or spirals at ends as indicated in NAVFAC P-272, Definitive Designs for Naval Shore Facilities, Drawing No. 1293323.

2.2.4.5 Impact. Forces induced by handling and driving shall be used with a load factor of 1.25 (allowable overstress of 33 percent).

2.2.4.6 Jetting. Where jetting is contemplated, the jet pipe shall be cast into the pile.

2.2.4.7 Class of Concrete. The class of concrete required is 5,000 psi (34.47 MPa) minimum for prestressed concrete, and 4,000 psi (27.58 MPa) minimum for nonprestressed concrete.

2.2.4.8 Standard Details. For information on standard details, refer to NAVFAC P-272.

2.2.4.9 Minimum Residual Prestress. The minimum residual prestress shall be 700 psi (4.83 MPa).

2.2.4.10 Minimum Wall Thickness (Piles With Voids). Provide a minimum of 1-1/2 in. (3.81 mm) clear cover on inside face (surface of void). In no case should wall thickness be less than 4 in. (101.6 mm).

2.2.4.11 Venting. If a void is provided which extends through to the lower end of the pile, vent the pile head to prevent the buildup of internal hydraulic pressure during driving.

2.2.4.12 Tolerances. Voids, when used, shall be located within 3/8 in. (9.5 mm) of the position shown on the plans. The maximum departure of the pile axis from a straight line, measured while the pile is not subject to bending forces, shall not exceed 1/8 in. (3.17 mm) in any 10 ft (3.05 m) length or 3/8 in. in any 40 ft (12.2 m) length. Overall sweep shall not exceed 0.1 percent of the pile length.

## 2.2.5 Cast-in-Place Concrete Piles

2.2.5.1 General. Cast-in-place concrete piles are not recommended for exposed locations and long unbraced lengths, especially when exposed to saltwater corrosion.

2.2.5.2 Casings. Casings shall meet the following criteria:

a) Casings shall have adequate strength to withstand the driving stresses and resist the distortion due to driving adjacent piles.

b) Except for portions of the pile embedded more than 5 ft (1.52 m) below the ground level, leave the casing above this level as sacrificial metal when estimating the structural capacity unless protective coatings or corrosion-resistant alloys are used. Metal of 1/8 in. (3.17 mm) or less in thickness shall not, in any case, be considered as contributing to structural capacity.

2.2.5.3 Minimum Tip Diameter. The minimum tip diameter of cast-in-place concrete piles shall be 8 in. (203 mm).

2.2.5.4 Reinforcement. Sections of piling which are above the point of fixity (as specified in para. 2.1.1.1) shall have sufficient casing thickness to provide residual metal at termination of design service life equal to minimum required reinforcement. Otherwise, such sections shall be reinforced with lateral ties and longitudinal bars in the same manner as precast piles. Cover requirements shall be as for precast piles. Spacers shall be provided for longitudinal reinforcement to assure that cover requirements are maintained.

2.2.5.5 Class of Concrete. The class of concrete to be used in saltwater shall be 4,000 psi (27.58 MPa) minimum, and for freshwater, 3,500 psi (24.13 MPa) minimum.

## 2.2.6 Steel H-Piles

2.2.6.1 Minimum Thickness of Metal. The thickness of metal shall be determined from consideration of loss of section as established in MIL-HDBK-1002/3, Steel Structures, unless corrosion protection is provided as described in para. 2.2.6.2. The minimum thickness shall not be less than 0.40 in. (10.55 mm). Splice plates shall not be less than 3/8 in. thick.

2.2.6.2 Corrosion Protection. When the required minimum thickness of metal is excessive, corrosion protection in the form of concrete, bituminous or plastic (epoxy) coatings, or cathodic protection shall be provided. When coatings are used, exercise special care to avoid damaging coatings during driving the piles. Bituminous or plastic coatings shall not be considered effective below the mudline and they require special care so they are not damaged through rubbing against the driving frame or template. In tropical environments, and other locations where corrosion is particularly severe, encase piling with concrete to 2 ft (0.61 m) below MLLW.

2.2.6.3 Cap Plates. Cap plates are not required for steel piles embedded in a concrete pile cap. Where structural design depends on bending in the piles

for stability, tie the tops of steel piles into the cap with reinforcing rods or structural sections welded to the pile and lapping the cap reinforcement.

2.2.6.4 Lugs, Scabs, and Core-Stopppers. Lugs, scabs, and core-stoppers may be used to increase the capacity of end-bearing steel piles. In fact, where H-piling sections are used as friction piles and where such piles are subject to severe impact loadings, it is desirable to fit the ends with such devices to prevent "driving" the piles under the action of the impacting load.

2.2.6.5 Hardware and Fittings. Refer to Section 4 for guidance on specifications for hardware and fittings for H-piling.

2.2.6.6 Limitation on Use. The tips of all steel H-piles having a thickness of metal less than 0.5 in. (12.69 mm), and driven to end-bearing on sound rock by an impact hammer, shall be reinforced. Observations of penetration resistance and the operation of the equipment shall be conducted so as to terminate driving directly when the pile reaches refusal on the rock surface.

## 2.2.7 Bell and Cylinder Piling and Screw Cylinders

2.2.7.1 Usage. Economical use of cylinder piles is normally attained where heavy moving loads must be supported, or where the piles must bear on rock or other hard bottom, at shallow depth, without sufficient overburden for lateral support.

2.2.7.2 On Hard Bottom (Rock or Hardpan). In general, a constant diameter shaft without the bell shall be used. Sloping surfaces should be leveled to receive the shaft. The need for anchorage in the form of dowels or keys shall be considered.

2.2.7.3 On Other Bottom Surfaces. Shaft and bell construction shall be considered if loads are heavy enough to warrant use of the bell. Except where supported on piling, embed the bell 2 to 4 ft (0.61 to 1.22 m) into firm material.

2.2.7.4 Minimum Reinforcement. Reinforcement shall be determined by design requirements, not by a minimum reinforcement ratio.

2.2.7.5 Embedment of Piling into the Bell. Embedment of piling into the bell shall be as required for transfer of load. Where tremie placement is employed, use 10 psi (69 kPa) bond resistance between pile and concrete, plus compression resistance of top of pile bearing in bell.

2.2.7.6 Protection for Reinforcement. The requirement for protection for reinforcement shall be the same as concrete piling, except it shall be reduced to 1-1/2 in. (38.1 mm) where permanent outer shell is provided.

2.2.7.7 Thickness of Metal Shell and Corrosion Protection. The specifications for thickness of metal shell, and corrosion protection, shall be the same as for H-piling.

2.2.7.8 Installation. Tremie placement of concrete fill is permitted. Provide for final cleanout of the bell or base of the cylinder immediately before concreting.

2.2.8 Steel Pipe Piles. Pipe piles shall conform to the applicable requirements for both steel H-piles and cast-in-place concrete piles (refer to paras. 2.2.5 and 2.2.6), except as modified in paras. 2.2.8.1 through 2.2.8.3.

2.2.8.1 Material. Material shall conform to ASTM A252, Welded and Steel Pipe Piles, unless otherwise approved.

2.2.8.2 Oven-End Piles. Pipes installed open end shall be resealed to full bearing after cleaning. If the pipe shell shows 2 in. (50.8 mm) or more of penetration on resealing, reclean and redrive in successive cycles until penetration on redriving is less than 2 in. If the leakage of water into the pile is minor, the pile shall be pumped out and one cubic yard of grout shall be placed before the balance of the concrete is installed. If the leakage of water makes it inadvisable to attempt to place concrete in the dry, then the shell shall be filled to its top with clean water.

The concrete is placed by the tremie method to the top of the pipe in one continuous operation or by using a grout seal of the same strength as the specified concrete. The grout seal, if used, shall be deposited by means of a grout pipe to an elevation of at least 3 ft (0.91 m) above the bottom of the pile. After a sufficient time has elapsed to allow the grout to set, the pile shall be pumped dry and the remaining space filled with concrete.

2.2.8.3 End Closure. For friction piles, end closures shall not project more than 1/2 in. (12.7 mm) beyond the outer limits of the pile shell.

2.2.9 Composite Piles. Size, capacity, and details of each section shall conform to the applicable requirements for each type of pile.

2.2.10 Sheet Piling--Steel. The requirements for steel sheet piling are the same as previously established for steel H-piling (refer to para. 2.2.6), except as modified in paras. 2.2.10.1 through 2.2.10.4.

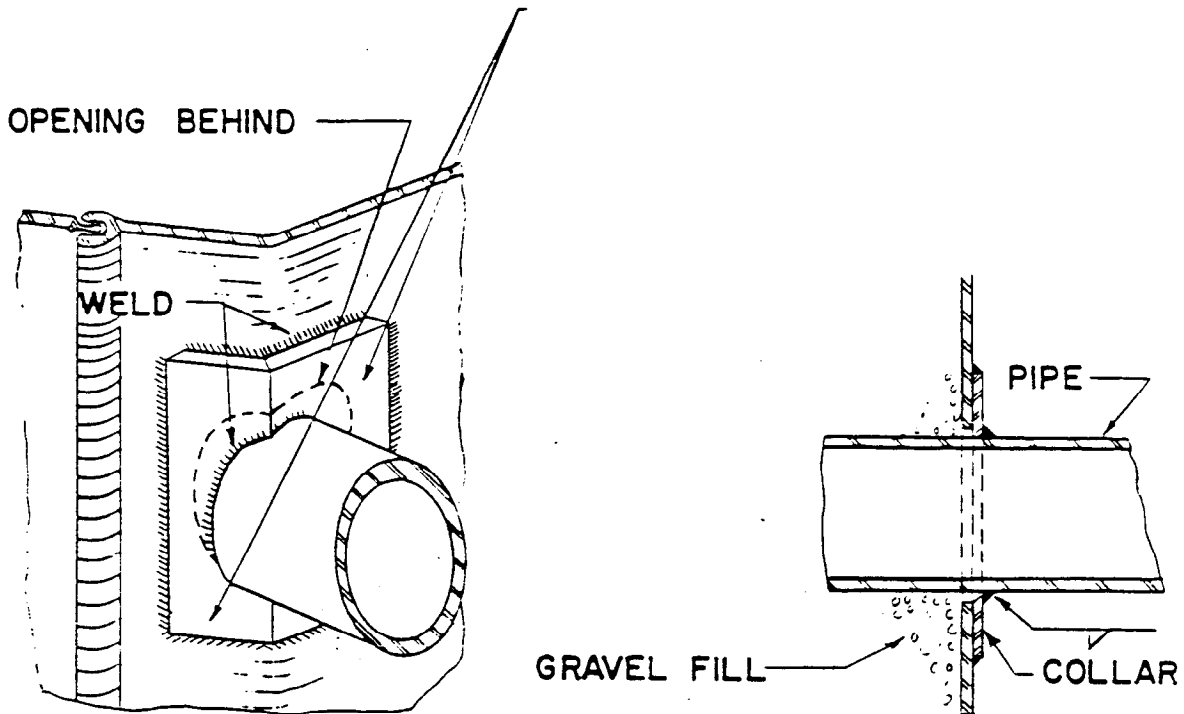
2.2.10.1 Splices. Splices shall consist of full-penetration butt welds. Splices, where the upper section of sheet is, driven as a follower to the lower section without positive connection between the two sections, will not be permitted even though splice may occur at a point of zero moment.

2.2.10.2 Connection to Cap. A 6 in. (52.3 mm) minimum embedment shall be used for concrete caps. The use of rods or structural sections for anchorage of sheet piling into the cap is not required. For steel channel caps, tack-weld each sheet to the cap.

2.2.10.3 Sleeves and Openings. All sleeves and openings for utilities passing through the sheets shall be detailed to prevent loss of fill (see Figure 2).

2.2.10.4 Minimum Thickness of Metal. For exposed face of cofferdams, 1/2 in. (12.7 mm) minimum shall be used. Elsewhere, a thickness consistent with design service life shall be provided. Thickness less than 3/8 in. (38 mm) may be used, but must be justified.

2.2.11 Sheet Piling - Concrete. Piling shall conform to the requirements stated above for precast concrete piles (refer to para. 2.2.4) except as



NOTE: ADD FILTER CLOTH OR SCREEN TO PREVENT LOSS OF FINES (WEEP HOLES ONLY).

Figure 2  
Sleeve Details

modified in paras. 2.2.11.1 through 2.2.11.4.

2.2.11.1 Joints. Joints shall be flushed and grouted and shall be tight to the mudline. Use of plastic sleeve is recommended (see Figure 3).

2.2.11.2 Ties Spirals or ties for longitudinal reinforcement are not required, except at tip and driving ends.

2.2.11.3 End of Sheets. Sheets shall be cast with a drift-sharpened point (see Figure 3). Embed tops of sheets 6 in. (152.4 mm) into a continuous cap.

2.2.11.4 Sleeves and Openings. The treatment of sleeves and openings in concrete sheet piling is similar to the procedure followed with steel sheet piling (refer to para. 2.2.10).

2.2.12 Sheet Piling--Timber. Timber sheet piling shall conform to the requirements in para. 2.2.3 for treated and untreated timber piles except as modified in paras. 2.2.12.1 through 2.2.12.5.

2.2.12.1 Treatment. Timber sheet piling shall bear the AWPB Quality Mark MLP. The types of treatment shall be as described for treated timber piles.

2.2.12.2 Joints. Joints shall be tongue and groove, or splined (or Wakefield Type sheeting may be used). Sheet piling shall be tight to the mudline. Tongue and groovesheets and splines shall have a loose fit.

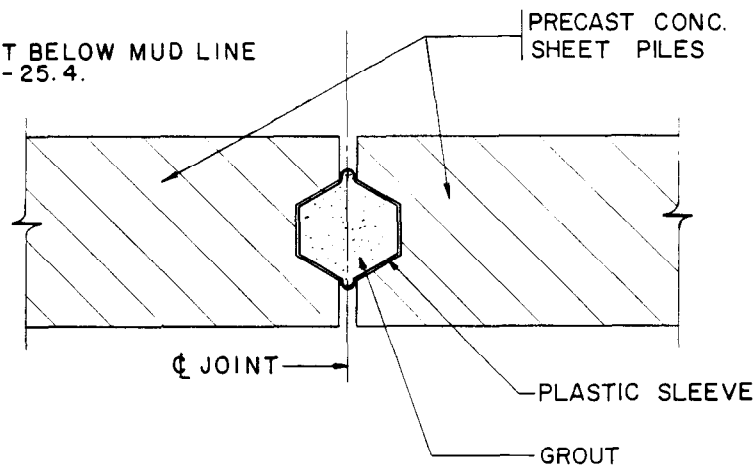
2.2.12.3 Drift Sharpening. Timber sheet piling shall be drift sharpened.

2.2.12.4 Tops of Sheets. Tops of sheets shall be drift-bolted or spiked to a continuous timber cap. Width of the cap shall be equal to or greater than the thickness of the sheet piling. Thickness of the cap shall not be less than 2 in. (50.8 mm). Where a concrete cap is used, embed sheets 6 in. (152.4 mm) into the cap.

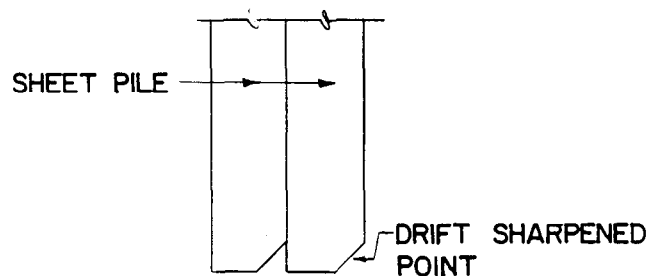
2.2.12.5 Sleeves and Openings. The procedure for incorporating sleeves and openings in the use of timber sheet piling is similar to that with steel sheet piling (refer to para. 2.2.10).

**NOTE:**

FOR DETAIL OF JOINT BELOW MUD LINE  
SEE FIGURE 15, DM-25.4.



**ABOVE MUD LINE  
JOINT TREATMENT**



**DRIFT SHARPENED POINT**

Figure 3  
Concrete Sheet Piling Details

## Section 3: DECK AND SUBSTRUCTURE FRAMING AND BRACING

3.1 Substructure. The substructure shall include pile caps, under-deck bracing, and other structural members (other than stringers) at and below the level of the pile caps.

3.1.1 Pile Caps - All Types. The effects of differential settlements of piles shall be investigated where:

- a) heavy concentrated loads occur,
- b) piles are long,
- c) there is an appreciable variation in pile lengths, and
- d) pile types or installation methods vary.

Differential settlement will not appreciably affect ultimate strength if the cap can yield without buckling or fracture.

3.1.2 Timber

3.1.2.1 Hardware and Fittings. For requirements pertaining to hardware and fittings, refer to Section 4.

3.1.2.2 Species and Preservative Treatment. Except for temporary structures, substructure timbers shall be given a preservative treatment and shall be a species which will accept deep treatment such as southern pine or douglas fir. For permanent structures, use of untreated timber is discouraged. Resistance to borers and decay of any untreated lumber--even of species presumed to be of superior resistance--is still inadequate for long term use. Species that do not accept preservative treatment shall be encased, or shall be used with full awareness of the need for maintenance.

3.1.2.3 Seasoning. Only seasoned timber shall be used for framing.

3.1.2.4 Minimum Dimension. A minimum of 3 in. (76.2 mm) (nominal size) in and below splash zone shall be used. Above the splash zone, the minimum size shall be 2 in. (50.8 mm) (nominal).

3.1.2.5 Retention and Penetration of Preservative. For guidance on the use of preservatives, conform to requirements of AWPB Standard MLP. The type of treatment used for treating timber piles shall be used.

3.1.3 Concrete

3.1.3.1 Cover. The cover shall conform to the requirements of para. 2.2.4.2.

3.1.3.2 Chamfer. The minimum chamfer for all corners shall be 3/4 in. (19.05 mm).

3.1.3.3 Class of Concrete. Concrete classes shall be as follows:

a) Precast concrete and concrete in a saltwater environment shall be 4,000 psi (27.58 MPa) minimum.

b) For other types and applications, use 3,000 psi (20.68 MPa) minimum.

3.1.3.4 Dimensional Changes. Concrete undergoes dimensional changes due to temperature and shrinkage and tends to swell if cast in-the-dry and then immersed in water. If such changes are prevented, stresses develop in the concrete. To minimize such stresses the following construction details should be observed:

a) Where thin concrete sections abut massive sections, allow for differential movements or break contact length into short segments by expansion joints.

b) Where new concrete abuts old, allow for differential movements, or break contact length into short segments by expansion joints.

3.1.4 Steel. For incorporating steel substructures, conform to the requirements for steel H piles (refer to para. 2.2.6).

3.2 Deck The "deck" shall include treads, planks, slabs, stringers, and other elements supported by the pile caps.

3.2.1 Timber. Timber used in the deck structure shall conform to the requirements for substructure framing and bracing except as modified below.

3.2.1.1 Treatment. Except for conditions described in a) and b) below, deck framing and bracing shall be given a preservative treatment. Do not use creosote treatment on walking surfaces or surfaces which normally will be touched by people (handrails, for example). Treatment is not required for:

a) Temporary structures, or

b) Timbers above Mean High Water (MHW) level, provided that resistant species (generally hardwoods) are used and the construction is detailed to provide for circulation of air around the timber and to minimize the extent of faying surfaces.

3.2.1.2 Hardware and Fittings. For information on hardware and fittings refer to Section 4.

3.2.1.3 Treads. Treads used in the deck structure shall conform to the following materials and dimensions:

a) Oak, maple, birch, black gum, or other species resistant to wear, are preferred.

b) Treads shall not be over 12 in. (304.8 mm) wide.

c) Treads shall not be less than 3 in. (76.2 mm) thick (nominal size).

d) A minimum of 3/8 in. (9.53 mm) clearance between treads shall be provided.

e) Preferably, attach treads to planks with drive screws; if treads are nailed, a minimum size of 20 penny nails shall be used.

f) Although treads cannot be considered as load carrying members, they can be considered in evaluating distribution of load to the planks.

3.2.1.4 Planks. Planks shall conform to the following criteria:

a) Planks shall be 12 in. (304.56 mm) wide (max.).

b) A minimum clearance of 3/8 in. shall be provided between planks.

c) Preferably, attach planks to stringers or nailers with drive screws. Where nailed, a minimum size of 20 penny nails shall be used.

3.2.1.5 Details. The following details shall be adhered to when timber is used in deck structures:

a) Solid bridging shall be provided between stringers at points of bearing and at intermediate lines at 20-ft (6.10 m) maximum spacing.

b) Stringers shall bear full width on caps. Adjacent stringers shall overlap caps.

3.2.2 Concrete

3.2.2.1 General. Conform to requirements for substructure framing and bracing, except that cover for reinforcement shall conform to requirements of DM-2.04 for normal exposure conditions.

3.2.2.2 Deck Finish. The deck shall be broom-finished to provide a skid-resistant surface.

3.2.3 Steel. When steel is used, it shall conform to the requirements for steel H piles (refer to para. 2.2.6).

3.2.4 General Provisions for All Types of Materials

3.2.4.1 Safety Provisions. The following safety provisions shall be adhered to:

a) Curb logs shall be provided to prevent vehicles from driving off deck. Curbs should be a minimum of 10 in. (254 mm) high (preferably 12 in. high) by whatever width is required for strength. Scuppers shall be provided as required. The minimum size of scuppers shall be 2 in. (50.8 mm) by 8 in. (203 mm).

b) For nonberthing faces, railing (fixed or removable) shall be provided for the safety of personnel.

3.2.4.2 Drainage. Deck drainage shall conform to the following requirements:

a) Pitch deck slab at a minimum of 1/16 in. per ft (1.59 mm per 0.3048 m) to drain to scuppers or collection points.

b) Where scuppers are permissible and feasible, use them in preference to drain holes. Size scuppers as described in para. 3.2.4.1 a).

c) Where drain holes are required, size as required for local rainfall intensity (25 year storm) with minimum diameter of 4 in. (101.5 mm). Locate drain holes between each pair of pile bents and at 20 to 30 ft (6.10 to 9.14 m) spacing parallel to pile bents.

d) Provide additional drain holes in service pits and in recesses for mooring fittings.

e) Provide 1-1/2 in. (38.1 mm) diameter drains in rail slots, two between each pair of pile bents.

3.2.4.3 Special Drainage for Petroleum Offloading and Fueling Piers. The following requirement shall be adhered to pertaining to special drainage for petroleum offloading and fueling piers:

a) An intercept system shall be required to collect oil spills. In normal operation, deck drainage shall outfall through the sump pumps into the harbor. If an oil spill occurs, pressing a deck-mounted button shall close a motor-operated outfall valve and start the sump pumps which pump the spill to a collection point. When the spill drainage procedure is completed and all oil is removed from the system, the system shall revert to normal operation.

b) In some cases, the contaminated rainwater runoff of all deck drainage (due to contact with residual drippings on the deck) shall be collected.

## Section 4: HARDWARE AND FITTINGS (PERMANENT INSTALLATION)

4.1 Saltwater--In or Below Splash Zone

4.1.1 Minimum Diameter of Bolts. The minimum diameter of bolts shall be 1 in. (25.4 mm).

4.1.2 Minimum Thickness of Metal in Straps and Fittings. The minimum thickness of metal in straps and fittings shall be 1/2 in. (12.7 mm).

4.1.3 Coatings. All hardware and fittings shall be galvanized except that galvanizing of ogee washers shall be optional.

4.1.4 Washers. All bolts shall be provided with ogee washers. Plate washers shall not be used without special approval of NAVFAC. In general, not more than two washers should be allowed under any bolt head or nut. Inclined bolts should be fitted with beveled washers.

4.1.5 Size of Bolt Holes. All bolt holes in timber (other than holes for drift bolts) are to be drilled with a bit having a diameter 1/16 in. (1.58 mm) larger than the diameter of the bolt shank. Alignment of bolt holes shall allow insertion by tapping with a mallet. Driving or force fitting of bolts is not allowed. Holes for drift bolts shall be 1/8 in. (3.17 mm) less in diameter than the bolt diameter. All drill bits shall be kept sharp and feed rate controlled to produce shavings, rather than chips.

4.1.6 Locking of Bolts. Nuts shall be checked or welded, or other positive means of locking the bolts shall be provided.

4.2 Saltwater--Above Splash Zone. Requirements shall be as for installations in or below splash zone except as modified below:

a) The minimum diameter of bolts shall be 3/4 in. (19.05 mm).

b) The minimum thickness of metal in straps and fittings shall be 3/8 in. (9.53 mm).

c) The use of ogee or plate washers shall be optional. The minimum thickness of plate washers shall be 1/4 in. (6.35 mm).

4.3 Freshwater. Requirements are the same for installation in saltwater, in or below splash zone, except as modified below:

a) The minimum diameter of bolts shall be 5/8 in. (15.87 mm).

b) The minimum thickness of metal in straps and fittings shall be 1/4 in. A minimum thickness of 3/8 in. (9.5 mm) is preferable.

c) The use of ogee or plate washers is optional. The minimum thickness of plate washers shall be 1/4 in.

4.4        Special Applications

4.4.1      Stainless Steel Fittings.    Stainless steel fittings shall be used for special applications, if cost permits.

4.4.2      Through Bolts.    Through bolts shall be used to the maximum extent feasible, for ease of replacement.

## Section 5: SPECIAL CONSIDERATIONS

5.1 Service Life. The provisions of MIL-HDBK-1002/1, Structural Engineering: General Requirements, relating to service life (25 years) shall apply except:

a) Estimating Service Life. Structures detailed in accordance with this handbook shall be assumed to meet the service life requirement in para. 5.1.

b) Accessibility. Since waterfront structures are commonly required to serve longer than 25 years, detail so that all areas or components with an anticipated service life of less than 50 years can be inspected and repaired.

c) Breakwaters. Breakwaters and other constructions whose design is controlled by wave forces shall be proportioned to resist a design wave of a 50-year interval of recurrence.

5.2 Corrosion of Steel Piling. The principal factors affecting rate of corrosion loss are:

- a) Geographical location,
- b) Zones relative to tidal planes,
- c) Exposure to salt spray,
- d) Sand, earth, or other cover,
- e) Protective coating,
- f) Abrasion conditions (surf zone versus deep water),
- g) Stray electric currents, and
- h) Type of soil.

5.2.1 Rate of Corrosion Loss. Refer to Table 1 for some experimentally derived corrosion loss rates. See Figures 4 and 5 for some actual, measured profiles. To estimate service life, refer to Table 2 for approximate periods of protection which can be expected to be afforded by various coating systems and estimate rate of loss of metal thereafter from Table 1, Figures 4 and 5, or from other available experience.

5.2.2 Tropical Climates. Steel piling (carbon-manganese steel) shall be faced with concrete in and above the tide range and to a minimum of 2 ft (0.61 m) below MLW. Steel sheet piling shall be capped with concrete and not with a steel channel or timber.

5.2.3 Use of Weathering Steels. The use of weathering steels shall conform to the following requirements:

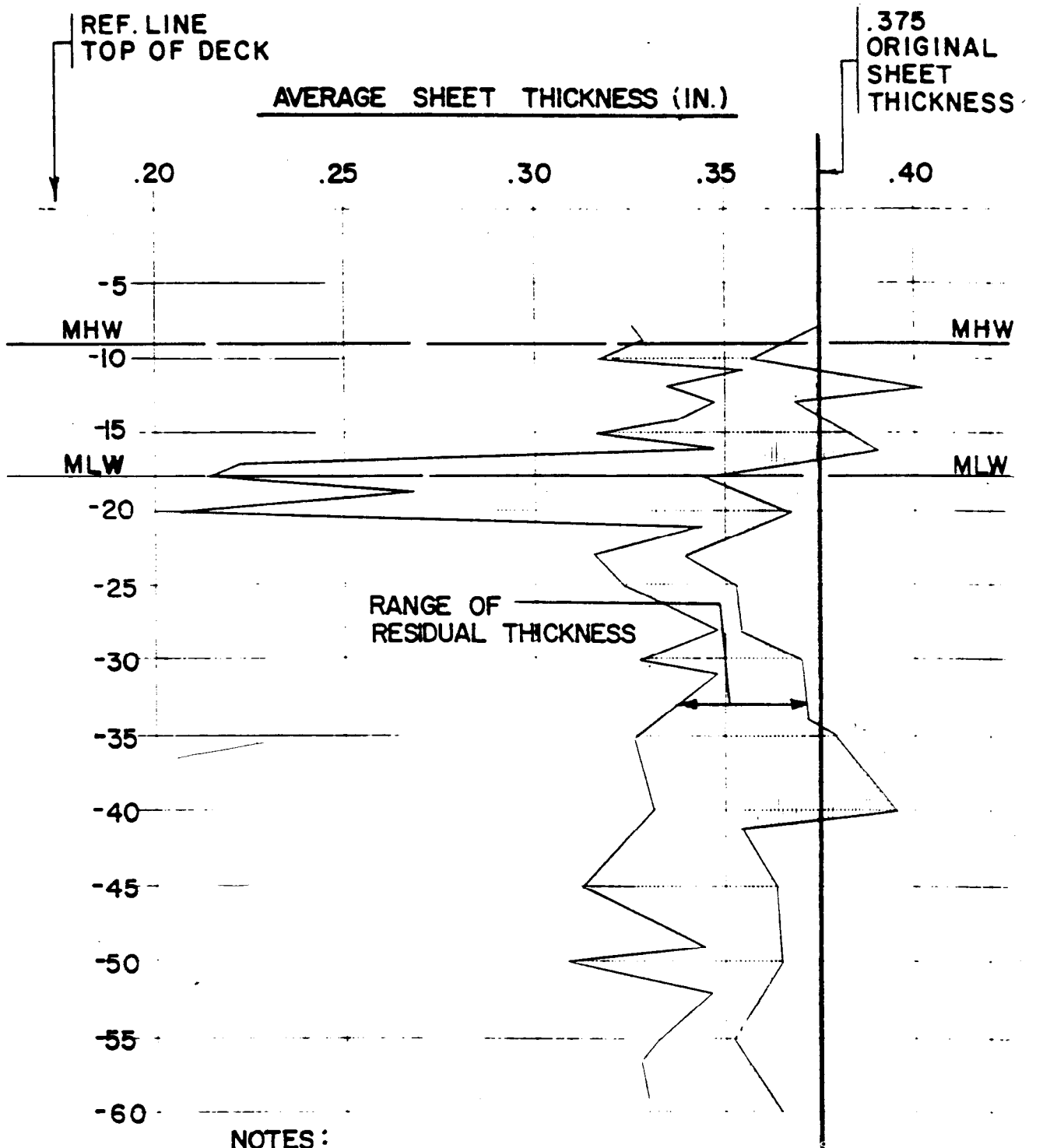
**Table 1**  
**Corrosion Rates of H-Piles Based On Flange Thickness Measurements(a)**

COATING DESCRIPTION	AVERAGE CORROSION RATE WITHIN ZONE (MILS PER YEAR)			
	IMBEDDED ZONE 0 to 15 ft (b)	EROSION ZONE 15 to 21 ft (b)	IMMERSED ZONE 21 to 29 ft (b)	ATMOSPHERIC ZONE 29 to 35 ft (b)
Coal Tar Epoxy/Zinc Rich Inorganic Vinyl/Flame Sprayed Aluminum Epoxy Polyamide/Zinc Rich Inorganic Aluminum Pigmented Coal Tar Epoxy Polyester Glass Flake Polyvinylidene Chloride/Flame Sprayed Zinc Galvanized Phenolic Mastic Flame Sprayed Aluminum Aluminum Pigmented Coal Tar Epoxy Coal Tar Epoxy/Zinc Rich Organic Vinyl/Zinc Rich Inorganic Vinyl Mastic/Zinc Rich Inorganic Coal Tar Epoxy Coal Tar Epoxy on Mariner Steel Coal Tar Epoxy plus Armor Coal Tar Epoxy Vinyl - Red Lead/Flame Sprayed Zinc Polyvinylidene Chloride Bare Carbon Steel Bare Carbon Steel Bare Carbon Steel	< 0.01 0.01 0.02 0.07 < 0.10  0 0 0.11 0.19 0.18 0.17 0.19 0.02 0.17 0.18 0.13 0.27 0.08 0.81 0.9 1.8 2.8	< 0.01 0.17 0.22 0.06 < 0.10  0.14 0.67 0.11 0.39 0.08 0.15 0.22 1.4 0.21 0.44 0.07 0.72 3.2 4.9 8.9 9.7 10.5	< 0.01 0.07 0.10 0.08 < 0.10  0.12 0.32 0.15 0.19 0.21 0.21 0.18 0.61 0.27 0.45 0.07 0.46 1.8 3.6 6.7 7.9 9.0	< 0.01 0 0 0.03 < 0.10  0.29 0.06 0.21 0.03 0.04 0.24 0.31 0 2.1 1.6 2.7 2.9 2.3 3.5 10.5 12.2 13.9

a Location: Atlantic Ocean, Dam Neck, Virginia.

Piles were 35 feet in length with 19 feet buried below mudline (sand). MLW was 4 feet above mudline and MHW 10 feet above mudline.

b Distance from bottom of pile.

**NOTES:**

LOCATION - CASCO BAY, MAINE

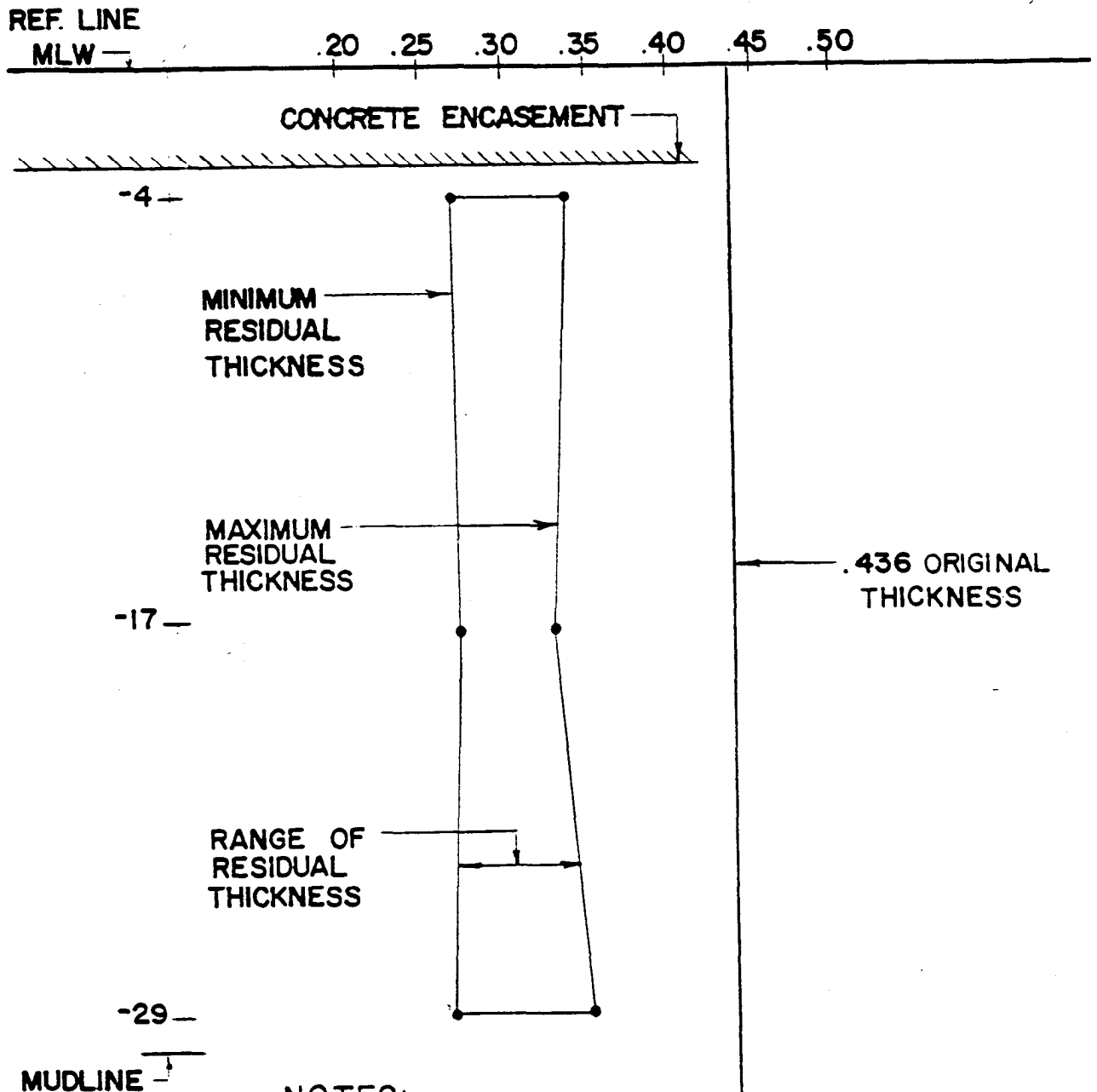
STRUCTURE - FILLED CELLS

AGE OF STRUCTURE - 27 YEARS

PROTECTIVE COATING - COAL TAR EPOXY, ABOVE EL. -20

NO COATING BELOW EL. -20

Figure 4  
Measured Profile Envelope--Steel Sheet Piling

**NOTES:**

LOCATION- CHARLESTON NAVAL SHIPYARD  
 STRUCTURE - PIER(AGE OF STRUCTURE- 37 YRS.)  
 PROTECTIVE COATING:  
 ABOVE EL. - 3 - CONCRETE ENCASEMENT  
 BELOW EL. - 3 - UNKNOWN

Figure 5  
 Measured Profile Envelope--Steel H-Piling

Table 2  
Period of Protection for Steel  
to be Expected from Various Coating Systems of Common Use<sup>a</sup>

COAT DESCRIPTION <sup>b</sup>	PERIOD OF PROTECTION <sup>c</sup>
Coal tar epoxy (15 to 20 mils thickness)	10 - 20 years
Galvanizing (7 to 9 mils thickness)	10 - 15 years
Metallized Aluminum	15 - 20 years
Concrete Encasement	25 years

a Marine exposure

b Coatings applied properly

c Periods of good to excellent protection, i.e., negligible loss of metal.

a) The following steels require coating in the splash zone and other areas not boldly exposed to sun, wind, and rain: Alloy steels conforming to the American Society for Testing and Materials Specifications (ASTM) A242, High-Strength, Low-Alloy Structural Steel; A588, High-Strength, Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4-in. Thick; or A690, High-Strength, Low-Alloy Steel H-Piles and Sheet Piling for Use in Marine Environments; and copper bearing steels conforming to ASTM A709, Structural Steel for Bridges.

There are no data on the rate of corrosion loss from surfaces in contact with the soil, so that consideration should be given to coating surfaces in the same manner and degree for carbon manganese steel (refer to ASTM A36, Structural Steel).

b) If an alloy conforming to ASTM A690 is used, hardware shall conform to ASTM A588.

5.3 Cathodic Protection. The desirability of providing cathodic protection in accordance with MIL-HDBK-1004/10, Cathodic Protection, for waterfront structures requires careful consideration, including the factors described in paras. 5.3.1 through 5.3.4.

5.3.1 Efficacy. In general, the rate of corrosion loss below MLW is two-thirds to one-half the rate just below, at, and above MLW. Since cathodic protection is effective only below MLLW, it follows that cathodic protection should be accompanied by use of concrete facing or encasement to and below MLLW. Alternatively, future repair by partial jacketing shall be considered in the economic analysis.

5.3.2 Maintenance Cost. The cost of electricity, replacement of anodes, and general repair of damage to wires and hangers shall be considered in the economic analysis.

5.3.3 Reliability of Maintenance Effort. A fully or partially inoperative cathodic protection system due to a lack of maintenance and repair is all too common. Routine scheduled maintenance inspections shall be implemented to minimize risk of failure of the cathodic protection system.

5.3.4 Coating Systems in Lieu of Cathodic Protection. Because highly effective coating systems exist, the use of coatings, in lieu of cathodic protection, may be considered. For structural applications, it is the overall loss of sectional area or section modulus that is important; pitting is not a concern. Accordingly, objections to certain types of coatings (i.e. holidays and small damages) which would be valid regarding their use for pipelines are not significant for piling.

5.4 Effects of Bulbous Bows and Projecting Propeller Guards. Many modern vessels have a bulbous bow. Theoretically, with careful maneuvering, these protuberances should not strike the fender system or project under the pier. However, their effect shall be considered in the design of the fender system, in the design and location of the outboard line(s) of piles, and in the design of a berthing bulkhead. Where feasible, the best solution is to overhang the fascia or to use camels. Figure 6 and Table 3 show the amount of underrun of the bulbous bow of various commercial vessels approaching the berthing face at various angles.

Although the projections of destroyers, cruisers, and frigates are greater in the lateral direction than those shown in Figure 6, underrun is not of concern for these naval vessels because the projections are located below and behind the waterline bow profiles. The exceptions are frigates of the FF 1040 and FF 1052 classes, which have projections extending laterally beyond the hull. For projections of propeller guards and stem planes for submarines, refer to NAVFAC DM-26.6, Mooring Design Physical and Empirical Data. In general, submarines are berthed using camels so that the underrun problem is not of concern.

5.5 Forces Due to Current and Propeller Swash. Current forces are normally neglected in the design of harbor structures. However, the rational design of exposed piling as a column, as described in Section 2, requires that lateral forces due to current be considered. The following formula may be used when estimating current forces:

**EQUATION:** 
$$P = \frac{(1b-sec^2)}{ft^4} \times 1.5 V^2 \quad (1)$$

where:

P = Pressure in pounds per square foot (lb/ft<sup>2</sup>)

V = Velocity in feet per second (ft/s)

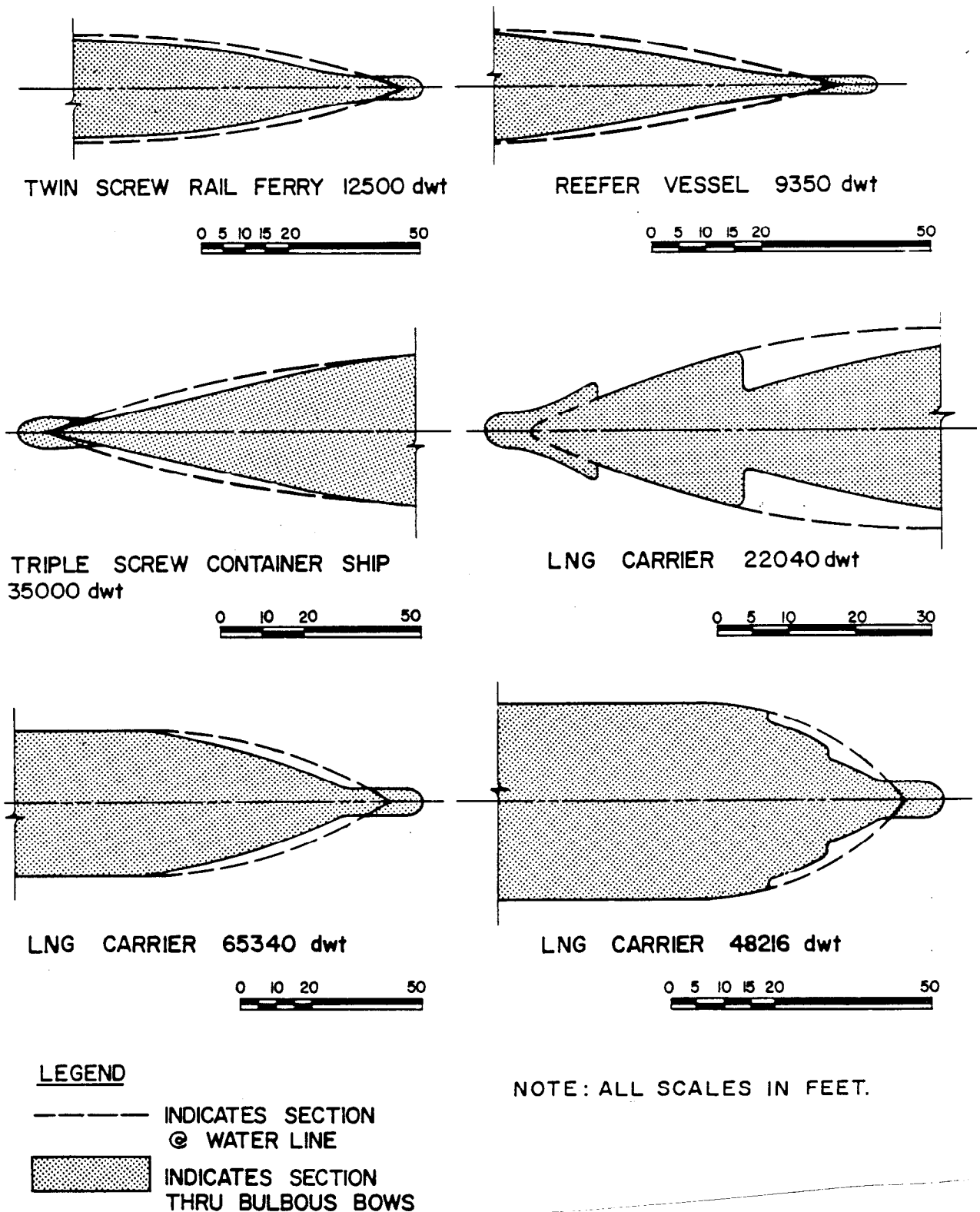


Figure 6  
Hull Configurations of Ships with Bulbous Bows

Table 3  
Extensions Under Pier (Bulbous Bows)

VESSEL	ANGLE OF APPROACH (DEGREES)			
	5	10	20	30
9350 dwt reefer vessel	-	0.62 m (2.03 ft)	2.5 m (8.20 ft)	3.72 m (12.20 ft)
12,500 dwt twin screw rail ferry	-		2.0 m (6.56 ft)	3.5 m (11.5 ft)
22,040 dwt LNG carrier	-	-	2.4 m (7.87 ft)	3.75 m (12.3 ft)
35,000 dwt triple screw container ship	-		2.6 m (8.5 ft)	3.0 m (9.85 ft)
48,216 dwt LNG carrier	-		-	1.75 m (5.75 ft)
65,340 dwt LNG carrier		-	2.75 m (9.0 ft)	5.5 m (18.0 ft)

dwt = deadweight tons

ft = feet

m = meters

LNG = liquefied natural gas

The coefficient 1.5 provides for roughness due to organic growth and for the resulting shape of the pile. Increase the effective diameter of the pile to compensate for the effects of organic growth. When designing the bearing piles within 20 ft (6.1 m) of a berthing face, assume a current velocity generated by the propellers of tugs or departing vessels to be 8 knots.

## 5.6 Expansion, Contraction, and Control Joints

5.6.1 Open-Pile Platforms. For information pertaining to open-pile platforms, refer to MIL-HDBK-1025/1, Piers and Wharves.

5.6.2 Bulkheads. In normal practice, no expansion or contraction joints are provided in the sheeting (whatever form it may take). However, expansion joints are provided in the concrete cap and encasement at approximately 30 ft (9.14 m) centers. If a timber or steel channel cap is used, no movement joints are provided, but adjacent sections of the cap are not connected. No movement joints are provided in the anchor wall.

5.6.3 Quaywalls. Movement joints shall be provided at approximately 300 ft (91.44 m) spacing. Such joints need not be carried through the sheet piling nor do they need to extend more than 5 ft (1.52 m) below MLLW.

## 5.7 Miscellaneous Requirements

### 5.7.1 General

5.7.1.1 Service Life. The actual service life of waterfront structures often far exceeds the specified design service life of 25 years. During this period, types and sizes of ships and usage change. One of the most important aspects of the design of berthing facilities is flexibility to accommodate multipurpose usage, such as:

a) A mix of sizes and types of ship (submarines and carriers are exceptions and often require dedicated berths), and

b) Nesting.

Another important consideration is to detail the design to provide ease of maintenance and repair of both the structure and the utility services.

5.7.1.2 Protective Lighting. The need for protective lighting in the following areas shall be considered. Lighting levels shall be as stated in OPNAVINST 5510.45B, U.S. Naval Physical Security Manual) for:

a) Land approaches to piers and docks.

b) Water approaches to piers and docks.

c) Decks of open piers.

d) Underside of platform decks of piers and wharves, including trestles.

5.7.1.3 Emergency Power. In general, any security area provided with protective lighting shall have an emergency power source located within that security area (refer to OPNAVINST 5510.45B).

5.7.1.4 NAVAIDS. NAVAIDS shall be provided at ends of piers, wharves, or quays. Cost of prominent, well-lighted markers is negligible compared to the cost and hazard of a collision. Refer to DM-26.1, Harbors, for specific requirements.

5.7.1.5 Fender System. Even though not intended for berthing, all piers, wharves, bulkheads, or quaywalls shall be provided with a fender system in consonance with the size of vessel that could be berthed (as limited by the depth of water and the length of the berthing face).

5.7.2 Structural. Waterfront structures shall conform to the following:

a) For bulkheads, where feasible, schedule dredging operations after the bulkhead sheeting is in place and tied back. For pile-supported platform structures, schedule dredging operations to precede the pile driving in order to minimize disturbance to piles already in place.

b) Unless positive restriction can be assured for a waterfront structure, assume that at some time during its service life a mobile crane

will be used on its deck. Even if not part of the project criteria, design for usage by a mobile crane of capacity specified in MIL-HDBK-1025/1 as applicable to the type of pier being considered. Use a load factor of 1.15 if rare and incidental use is intended, or higher load factors if more frequent use is judged to be probable.

c) Recess hold-down bolts in bases of mooring fittings to avoid interference with lines, and fill pockets with lead.

d) To the extent feasible, use through-bolts in sleeved holes for fastening mooring fittings and fender system components to structures.

e) Allowable tension in concrete in a seawater environment is the same as in an upland environment (refer to American Concrete Institute, ACI-318, Building Code Requirements for Reinforced Concrete).

5.7.3 Utilities. In designing waterfront utilities, layouts shall be made of all proposed and potential future services to check for fit, interferences, and accessibility. Provide for future installation of additional lines if the cost of doing so is not excessive. Additional size of trench, additional size in utility stations, room for expansion of substations, and--most important-- sleeves and clearance, should be considered. Include the assumed configuration of hoses and connections from utility stations on the structure to stations on the ship. Check to see if the weight of the hose or cable will require a jib crane, boom, or other special device for handling.

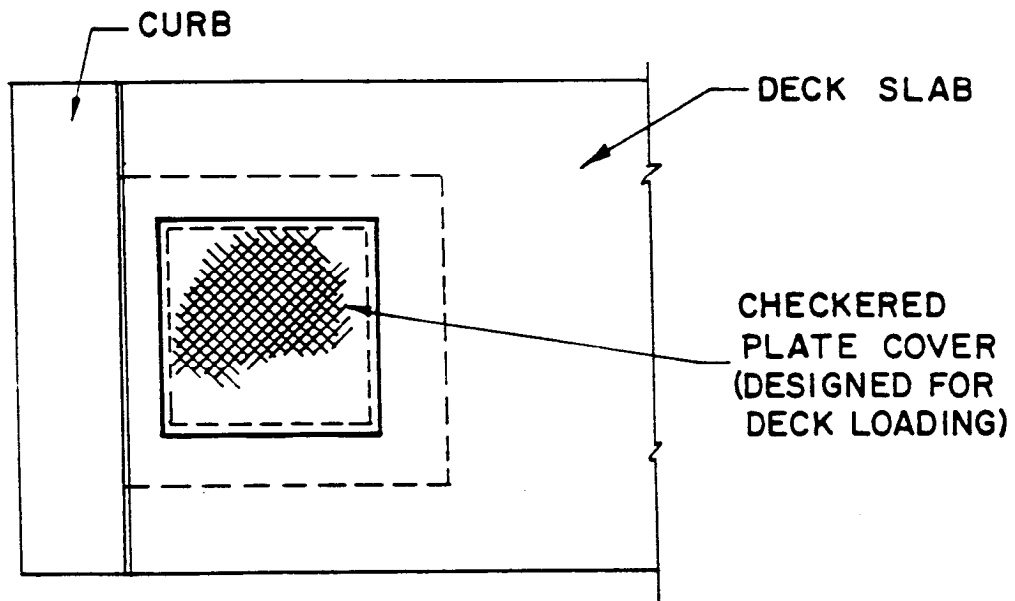
Security measures shall be provided to prevent unauthorized access to key items of equipment such as switchgear, fire pumps, pump wells, or caisson gates.

5.7.3.1 Abovedeck Lines. If feasible, do not place service lines below deck level unless in trenches or in ballast. If the use of trenches or ballast is impractical or grossly uneconomical, lines shall be protected from corrosion by high-performance coatings. Do not place service lines where they will be directly exposed to saltwater or to salt spray unless protected by coatings.

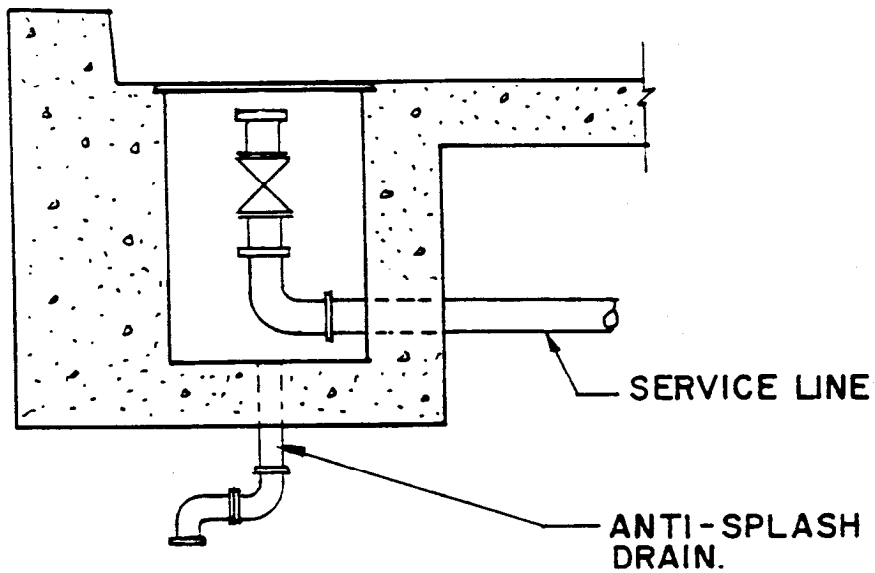
5.7.3.2 Underdeck Lines. Where exposed underdeck lines must be used, locate them behind protective piles or high enough to avoid impact damage from floating ice or debris. Do not place service lines where they will be subject to impact of drift. Work platforms and access shall be provided at underdeck valves and other operating fittings. For underdeck lines, drain valves shall be placed in accessible locations to avoid any necessity for climbing over the side of the platform.

5.7.3.3 Outlets, Connections, and Access Hatches. Outlets and access hatches in decks shall have flush-mounted covers and are to be detailed to eliminate any danger of tripping. Flush deck service outlets are shown in Figure 7. Where outlets and connections must protrude above the deck level, cover them in a manner that will ensure personnel safety and prevent any possibility of mooring lines being secured to the piping. See Figure 8 for suggested details. Do not place outlets or connections where they will be directly exposed to saltwater or salt spray unless protected by coatings.

5.7.4 Dredging Under Platforms. A set of soundings taken under an open

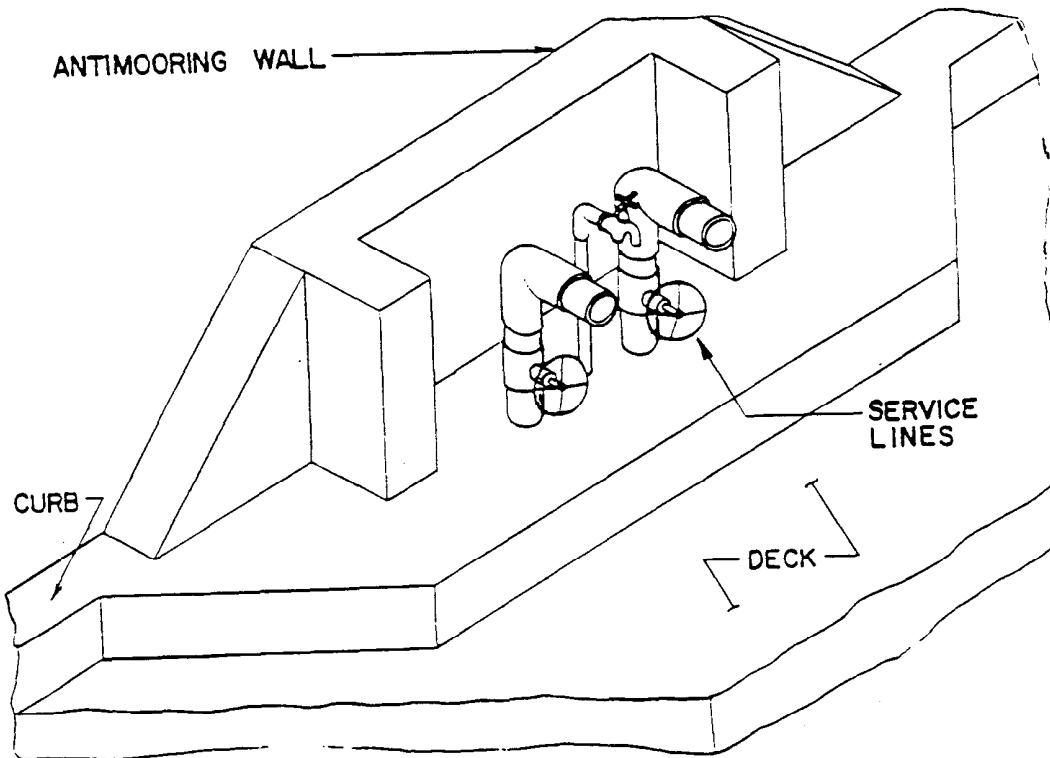


PLAN

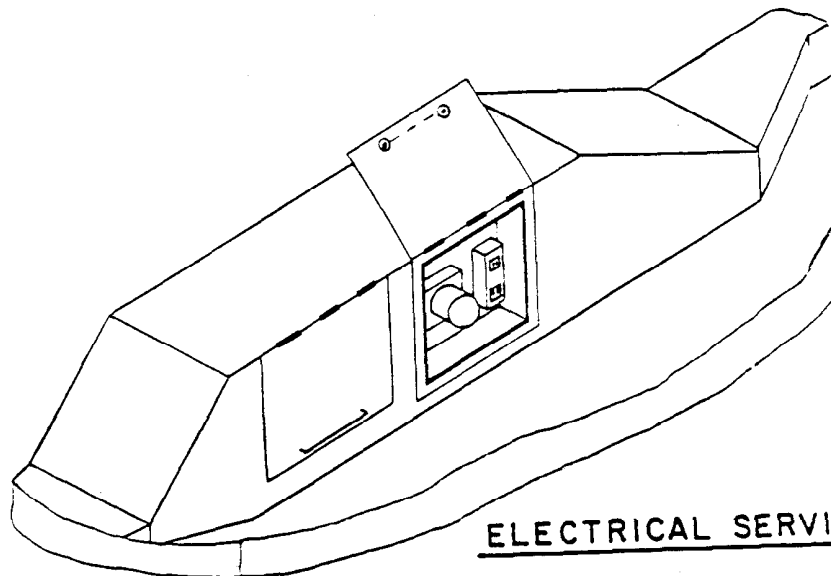


SECTION

Figure 7  
Flush Deck Service Outlets



MECHANICAL SERVICES



ELECTRICAL SERVICES

Figure 8  
Protection of Utilities Above Deck Level

pile platform shortly after dredging, or after some years of accretion of silt, typically looks like Profile A in Figure 9. Particularly for closely spaced bents, the restraint offered by the piles results in a slope steeper than the normal angle of repose. Eventually, this material will slough, or wash down into the slip. If the material is soft or loose, or for granular soils, it flows around the piles and no harm results other than additional maintenance dredging. The lateral loads on the piles due to movement of the soil are small. For cohesive soils, however, blocks of material may tend to move, or a mass failure (curve B of Figure 9) may develop, either of which can exert substantial lateral forces on the piles, which the piles are ill-suited to resist. Accordingly, at least for cohesive soils, washing down of the soil under the pier should be specified every few cycles of maintenance dredging. Initial dredging should include a controlled slope under the platform (not just dredging up to the fascia and letting the soil fall in), and the initial dredged slope should be flattened as shown in Figure 9 to meet the daylight line of probable future dredging.

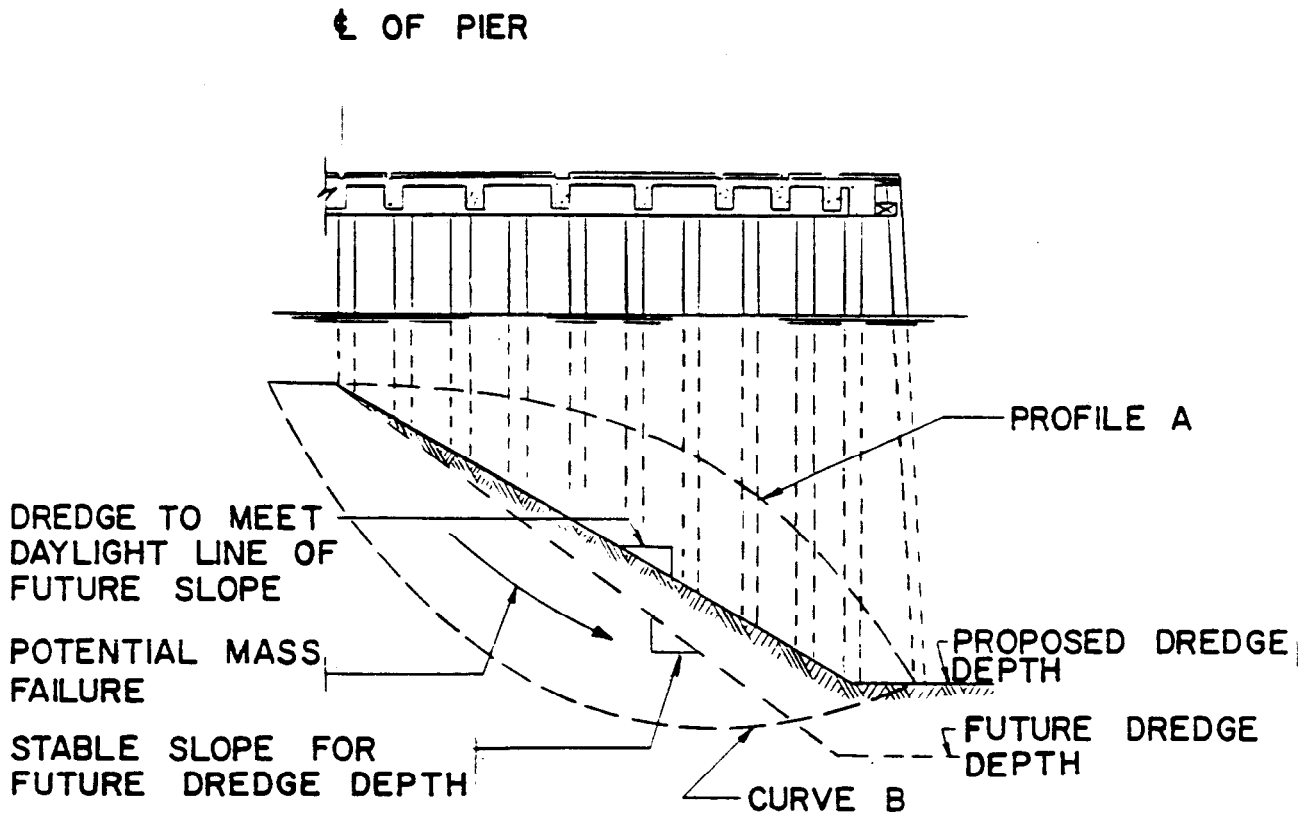


Figure 9  
Dredging Under a Platform

5.8 Detailing Fenderinn System to Resist Effects of Rolling of the Vessel. A fender system is designed to resist the impact of an approaching vessel. Commonly, the impacting blow is assumed to occur flat on to the fendering, i.e., assuming no list on the vessel. This is not always correct; when berthed alongside, the vessel rolls. The consequence of both of these events is that certain details, such as those shown in Figure 10, work poorly. A fender system should be detailed on the assumption of 5" roll or list of the berthed, or berthing, vessel.

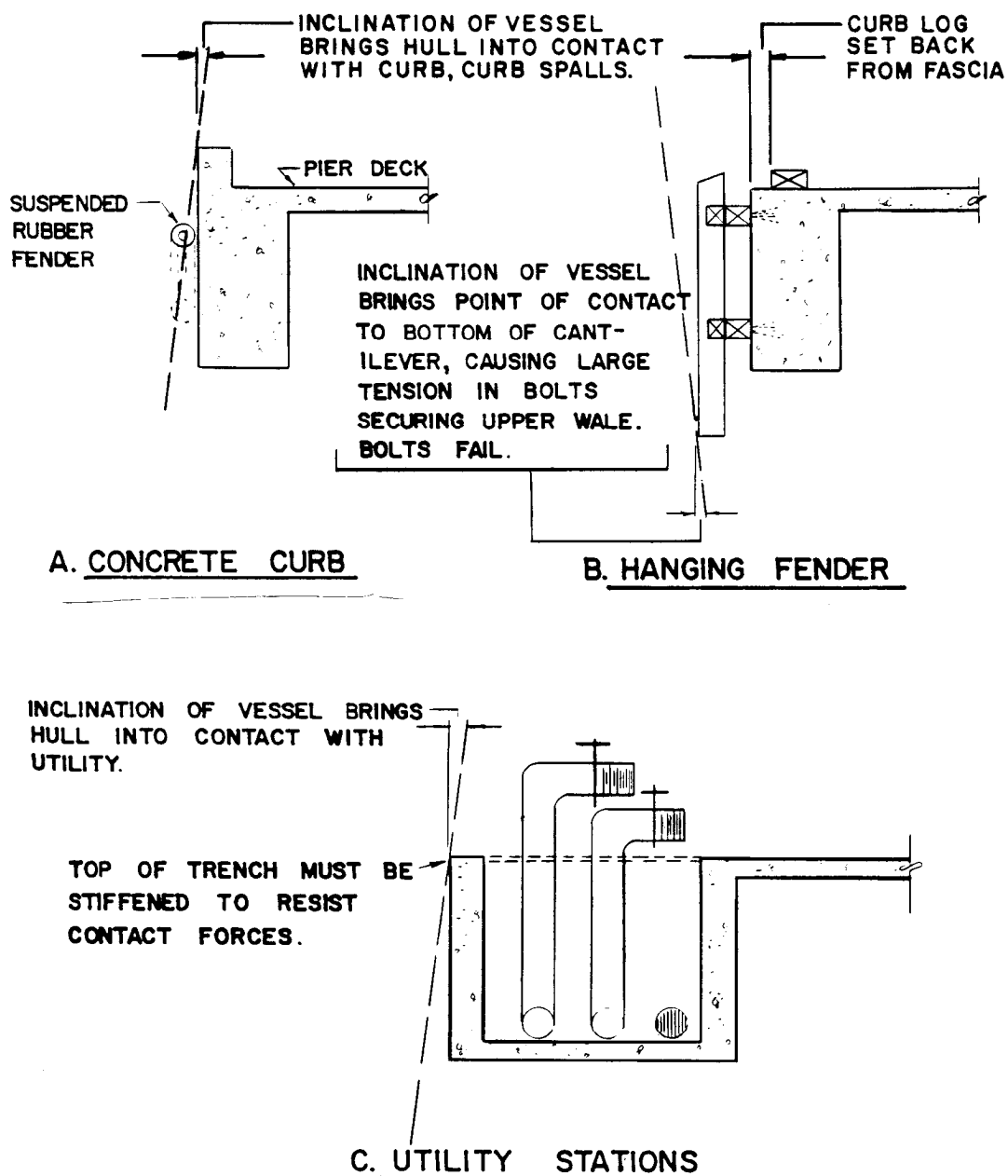


Figure 10  
Some Details Which are Affected  
By List or Roll of Berthing (or Berthed) Vessels

5.9 Strength of Wood Which Has Been Given a Preservative Treatment. For treated douglas fir or southern pine (piles or sawn timbers), reduce strain, strength, and energy absorption properties as indicated in Table 4.

#### 5.10 Fire Protection Requirements

5.10.1 General. The provisions of the National Fire Protection Association (NFPA) Standard NFPA 87, Construction and Protection of Piers and Wharves, and of MIL-HDBK-1008, Fire Protection for Facilities Engineering, Design, and Construction, shall apply. The following supplementary rules have been provided for ease of reference:

a) Do not rely on low pressure water connections as an adequate fire protection system.

b) The fire protection system shall be capable of use for fighting fires on and in vessels berthed at the pier, wharf, or bulkhead, as well as a fire on and under the structure.

c) Fire protection system shall be tailored to the type and condition of vessels being berthed. For example, a saltwater system would be inappropriate at most submarine berths.

d) Do not use flush deck-type valved outlets in locations subject to accumulations of ice and snow.

e) Where wood deck or wood piles are used, provide firewalls across the width of the pier at a maximum of 150 ft (45.72 m) intervals. Firewalls are to extend from the underside of the deck to MLLW.

f) Whatever the materials of construction (including noncombustible materials), provide 6 in. (152 mm) diameter holes (minimum) with removable covers and, where feasible, provide asbestos-cement liner in the deck to permit insertion of spray nozzles to fight below deck fires such as those due to floating, burning oil. Locate these holes on a 25-ft (7.62 m) maximum grid.

g) Detail the fender system to permit access at intervals not exceeding 150 ft (45.72 m) to fight floating fires. Access manholes in the deck are not desirable.

h) If a shed is provided, do not support the shed columns on timber grillages or piles unless the use of timber is restricted to areas below the level of permanent saturation (generally about midtide level). A concrete pedestal shall be built around the level of permanent saturation.

i) A standpipe line in a pier shed shall be provided so that firefighting equipment need not go into the pier shed to fight a fire.

j) Shotcrete or concrete jacketing to mean low water is recommended for wood piles.

Table 4  
Properties of Treated Woods

TYPE OF TREATMENT	AVERAGE PROPERTIES							
	MODULUS OF RUPTURE		MODULUS ELASTICITY IN FLEXURE		AVERAGE ABSORB. ENERGY IN FLEXURE		COMPRESSIVE STRENGTH $F_c$	
	psi	%	$10^6$ psi	%	in - lb/ in. <sup>3</sup>	%	psi	%
Fir								
Untreated	8,394	100	1.922	100	6.388	100	3,346	100
Creosote	6,862	82	1.584	82	4.202	66	—	—
ACA dual	6,111	73	1.537	80	3.059	48	2,714	81
CCA dual	3,844	46	1.171	61	3.364	53	2,333	70
ACA	5,620	67	1,416	74	2.078	33	2,462	74
Pine								
Untreated	8,007	100	1.942	100	5.240	100	—	—
Creosote	5,950	74	—	—	—	—	—	—
ACA dual	4,725	59	1.568	81	2.829	54	—	—
CCA dual	4,167	52	1.441	74	2.413	46	—	—
ACA	5,534	69	1.538	79	—	—	—	—
CCA	5,410	68	—	—	—	—	—	—

NOTES: 1) Where no value is provided, it is because of the large spread in measured values for a small number of samples.

2) % = the percent of the value for untreated wood.

3) Source: Civil Engineering Laboratory, Technical Note (TN) No. N1535 Mechanical Properties of Preservative Treated Marina Piles - Results of Limited Full-Scale Testing.

## Section 6: STRENGTH EVALUATION OF EXISTING WATERFRONT STRUCTURES

6.1 Evaluation of Strength of Existing Materials.

6.1.1 General. The provisions of MIL-HDBK-1002/1 relating to the use of used and unidentified materials shall apply.

6.1.2 Number of Tests Required to Establish Strength of Ungraded Materials. Where documentary data as to quality of the existing material is lacking, the strength shall be established by tests of the material. The strength of material to be assumed for strength evaluation of the structure shall be the value which sampling and test indicates to have a 95 percent probability of being exceeded. Not less than four samples shall be tested.

Strength Calculation. Calculate the strength of material for a given set of tests as:

$$\text{EQUATION:} \quad S = x - \frac{\sigma t}{2} \quad (2)$$

where:

- x** = average (arithmetic mean) of tests
- $\sigma$**  = standard deviation of the test values  
(A normal distribution curve may be assumed.)
- t** = a factor (see Figure 11)

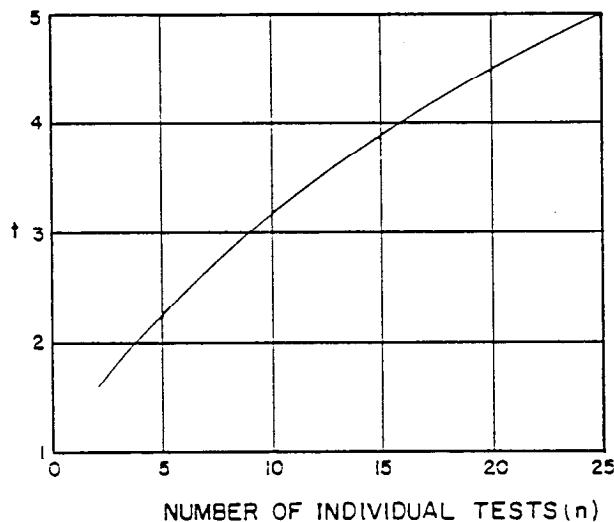


Figure 11  
Values of "t" for a Given Number of Individual Tests

### 6.1.3 Tests and Test Specimens

6.1.3.1 Steel Members. For steel members, take test specimens from locations and as described in ASTM A6, Standard Specifications for General Requirements for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use.

6.1.3.2 Concrete Members. For concrete members, use of drilled cores and sawed beams shall be as described in ASTM C42, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.

6.1.3.3 Wood Members. For wood members, stress-grade visually as described in ASTM D245, Standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.

6.2 Computation of Strength of the Structure. Analysis shall be based on measured in-place dimensions and as-built conditions. A badly deteriorated or obviously overloaded structure often continues to support the applied loads with no discernible indications of distress. It is important to consider the factors to this phenomenon when evaluating the strength of an existing structure. The more important of these factors are presented in paras. 6.2.1 through 6.2.8.

6.2.1 Simplifying Design Assumptions. Structural design commonly employs simplifying assumptions intended to make the design effort more manageable. These assumptions, necessarily, are conservative. Often, they leave substantial excess strength; for example, in presenting the distribution of concentrated loads to a slab, and in the structural design of variable sections.

6.2.1.1 Distribution of Concentrated Loads to a Slab. Conventional procedures, such as those described in the references of MIL-HDBK-1002/3, DM-2.04, and the American Association of State Highways and Transportation Officials (AASHTO), grossly underestimate this distribution. Use grid analysis (and computer) to obtain more realistic answers.

6.2.1.2 Variable Section. Figure 12 shows the cross section of a pier supported on steel H-piles. The upper portion of the piles was jacketed with concrete to protect against corrosion. The stiffening effect of the jackets reduced the slenderness ratio ( $L/r$ ) by 10 percent and increased the structural capacity of the piles by 25 percent.

6.2.2 Locations of Weakened Sections. Members are proportioned for maximum stress conditions. The section required at points of maximum stress frequently is carried for the full length of the member to minimize the costs of fabrication or of form work, or for aesthetic reasons. If the deterioration of a member is localized and does not occur at a point of maximum stress, the strength of the section may not be impaired. For example:

a) The piles shown in Figure 12 had suffered substantial corrosion loss in the level just below the bottom of the jackets. Loss of section up to 40 percent was measured. However, the magnitude of loss was limited to a distance of a few feet. Below this area, corrosion attack was less. Analysis of column capacity based on actual, measured sections indicated capacities

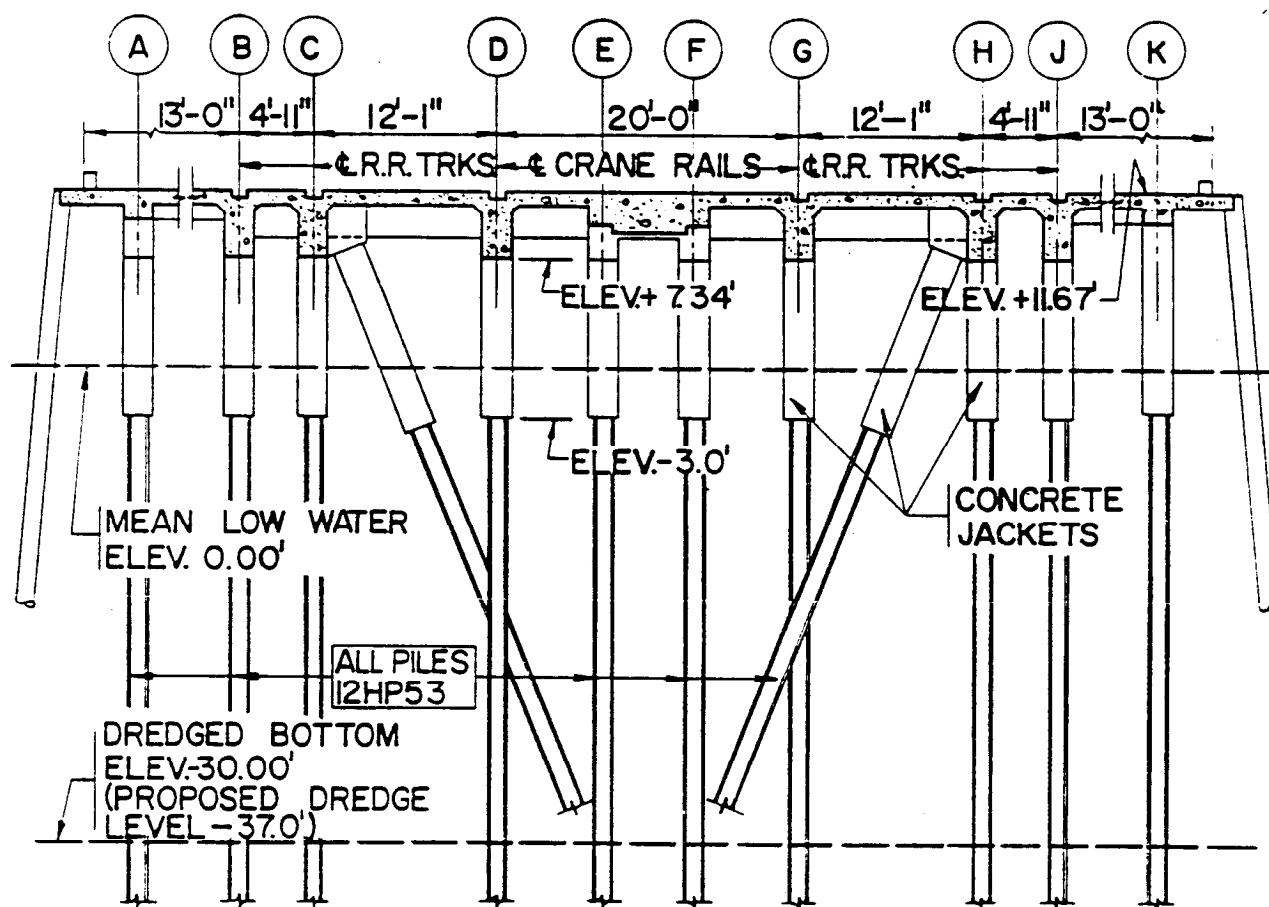


Figure 12  
Pier on Steel H-Piles

were 20 percent or more, greater than if the maximum loss had occurred over the full length of piles, or at the middle of the buckling length.

b) Figure 13 shows a stringer of a trestle. Leakage through the expansion joint over the abutment has corroded the flange until there is little section left. However, there is negligible moment at this section. Accordingly, the loss of flange material is not critical.

6.2.3 Changed Design Standards. If the design is based on an elastic analysis, reanalysis on the basis of ultimate strength, of plastic redistribution of moments, or of relief of moment based on the concept of the yield line frequently will indicate a greater capacity.

6.2.4 Design Live Loads. Design live loads are seldom realized in practice. The actual and the design loading conditions should be compared.



Figure 13  
Corrosion of a Stringer

6.2.5 Excess Section. The design may incorporate excess strength by way of sacrificial metal, by sizing the members to the next heavier section, or to a lighter but stronger section, or by satisfying requirements for minimum thickness of metal or limiting deflection. Piling, in particular, often is sized to resist driving stresses (or for load transfer to the soil). Static stresses are much less.

6.2.6 Increased Strength of Concrete With Age. An increase of 30 percent, after 2 years, compared to the 28-day strength, would be a reasonable expectation. The resulting increase in moment capacity would be about 6 percent.

6.2.7 Change of Structural Action. The structural action may differ from that assumed for purposes of design. Ordinary beams and slabs are a common case in point. These are proportioned on the basis of flexural behavior. However, except for large ratios of span to depth, pure flexural action is not

achieved, and the member resists the load at least partly, by catenary action (see Figure 14) or arching (see Figure 15). Composite action may develop, which was absent in the original construction (see Figure 16). Yield points may develop, changing the moment diagram and reactions, thus increasing some and decreasing others, with the changes often being noncritical (see Figure 17).

6.2.8 Change in Loading. In some cases, the design load represents a temporary or construction condition, and the service loads are of lesser magnitude. For example, consider a retaining wall. If the wall is well drained, maximum lateral pressure will occur during and shortly after backfilling with the active pressure decreasing with time. Another example is that of a hydraulic fill. The lateral pressure decreases as the fill drains. Borings will help in evaluating actual, in-place soil properties at the time that evaluation is made.

### 6.3 Evaluation of Concrete Strength By Use of Load Tests

6.3.1 Method. The provisions of Chapter 20 of ACI-318 shall apply, supplemented as described in paras. 6.3.2 through 6.3.4.

6.3.2 Test Load. Dimensions of  $1.4D + 1.7L$  shall be used where live load is the reduced load (for tributary area). This increased loading intensity will require careful observation and control to preclude precipitating collapse. For this purpose, load in six increments, rather than four, and where feasible, use water loading provision for emergency drainage.

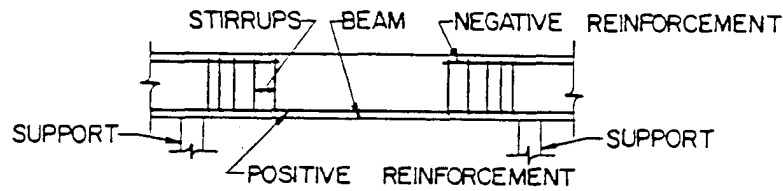
6.3.3 Lateral Loads. Lateral loads which are simultaneous with vertical loads shall be simulated in the test.

6.3.4 Loaded Area. The loaded area shall be large enough that reserve of strength due to continuity and three dimensional action of the structure is properly reflected in the test.

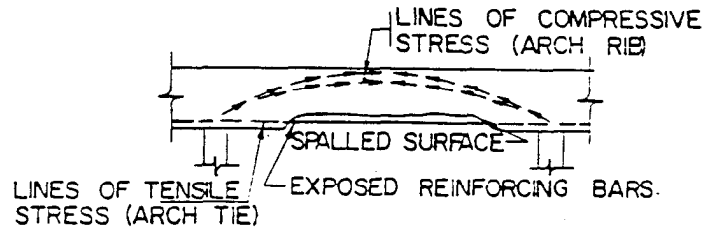
### 6.4 Special Provisions Regarding Capacity of Existing Piles

6.4.1 Structural Capacity. Existing piles shall be checked for effects of deterioration. A reconnaissance survey should be made to identify areas of "worst conditions." I Measurement of overall residual strength of 1 percent to 2 percent (but not less than 4) of the piles will be considered as an adequate statistical sample on which to base judgment of capacity. These shall be the "worst" piles of the group, as identified in the reconnaissance survey. Consideration shall be given to probable, future progression of loss of strength. Often the mudline under a platform has accumulated well above the normal, stable slope line drawn from the existing dredge level alongside the platform. This material shall be discounted in estimating the unbraced pile length (L). Should future dredging to greater depth be contemplated, consider the increased pile length that would result.

6.4.2 Capacity of the Soil to Support the Pile. Unless special circumstances prevail, for example, loss of support due to dredging, assume no change from capacity as installed. Where installed capacity is not known, consider the use of load tests to establish capacity.

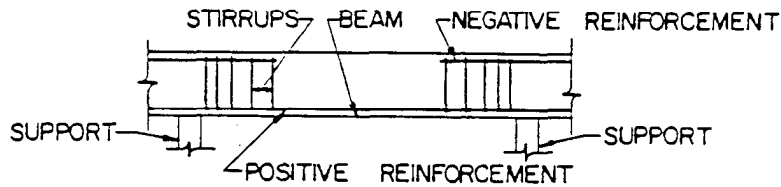


1) CONSTRUCTION

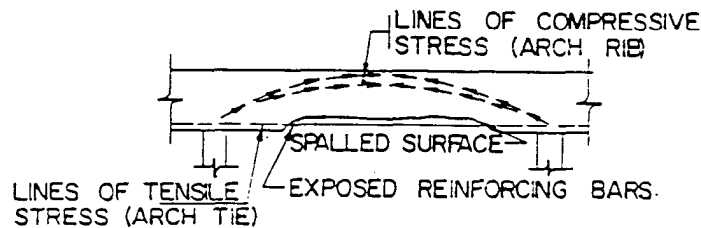


2) STRUCTURAL ACTION RESULTING FROM DEVELOPMENT OF DEFECTS

**Figure 14**  
Catenary Action Due to Cracking of Reinforced Concrete Member

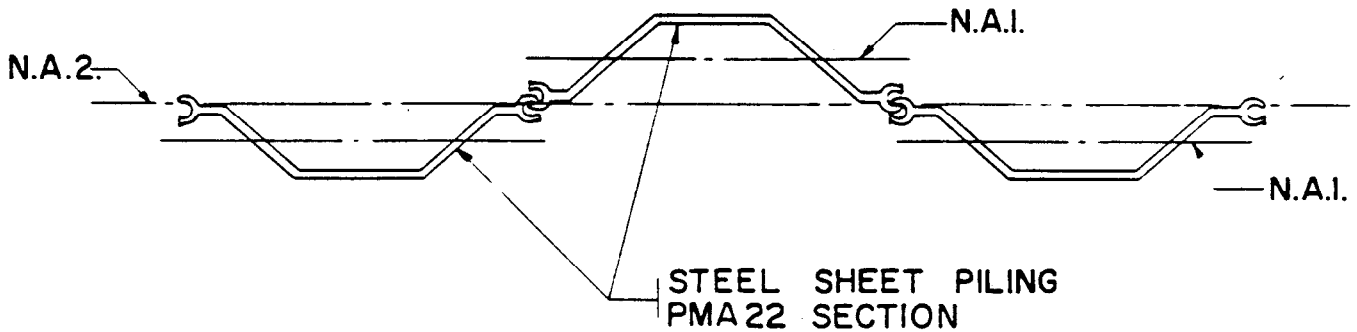


1) CONSTRUCTION



2) STRUCTURAL ACTION RESULTING FROM DEVELOPMENT OF DEFECTS

**Figure 15**  
Arch Action Due to Spalling of Reinforced Concrete Member



- NOTES: 1) DESIGN STRENGTH PREDICATED ON EACH SHEET PILE ACTING INDIVIDUALLY. SECTION MODULUS= $5.4 \text{ in}^3$  PER FOOT OF WALL (PMA 22 SECTION)
- 2) IF INTERLOCKS RUST TIGHT AND CAN DEVELOP SHEAR, NEUTRAL AXIS SHIFTS TO N.A.2 AND SECTION MODULUS IS INCREASED TO  $7.1 \text{ in}^3$  PER FOOT OF WALL, AN INCREASE OF 32%.
- 3) STEEL SHEET PILING HANDBOOK RECOMMENDS USE OF SECTION MODULUS OF  $6.8 \text{ in}^3$  PER FOOT OF WALL (PMA 22 SECTION).

Figure 16  
Increase of Strength of Sheet-Pile Wall Due to Aging

6.4.3 Sheet Piling. The capacity of sheet piling to support vertical loads shall be taken as one-half the value indicated by conventional formulae relating capacity to driving resistance.

6.4.4 Interpretation of Load Tests. For interpretation of load tests refer to DM-7.01, DM-7.02, and DM-7.03.

## 6.5 Strengthening an Existing Structure

6.5.1 Methods. Methods of strengthening existing structures are illustrated in Figure 18.

6.5.1.1 Plating. Where the top of flange is not accessible for adding cover plates, reinforcement can be added to the web plate. The beam shall be relieved of load before the reinforcement is added. When cover plates are added, the flange to web connection and the web plate stresses at the toe of the flange shall be investigated.

6.5.1.2 Composite Action. Beam section properties can be materially

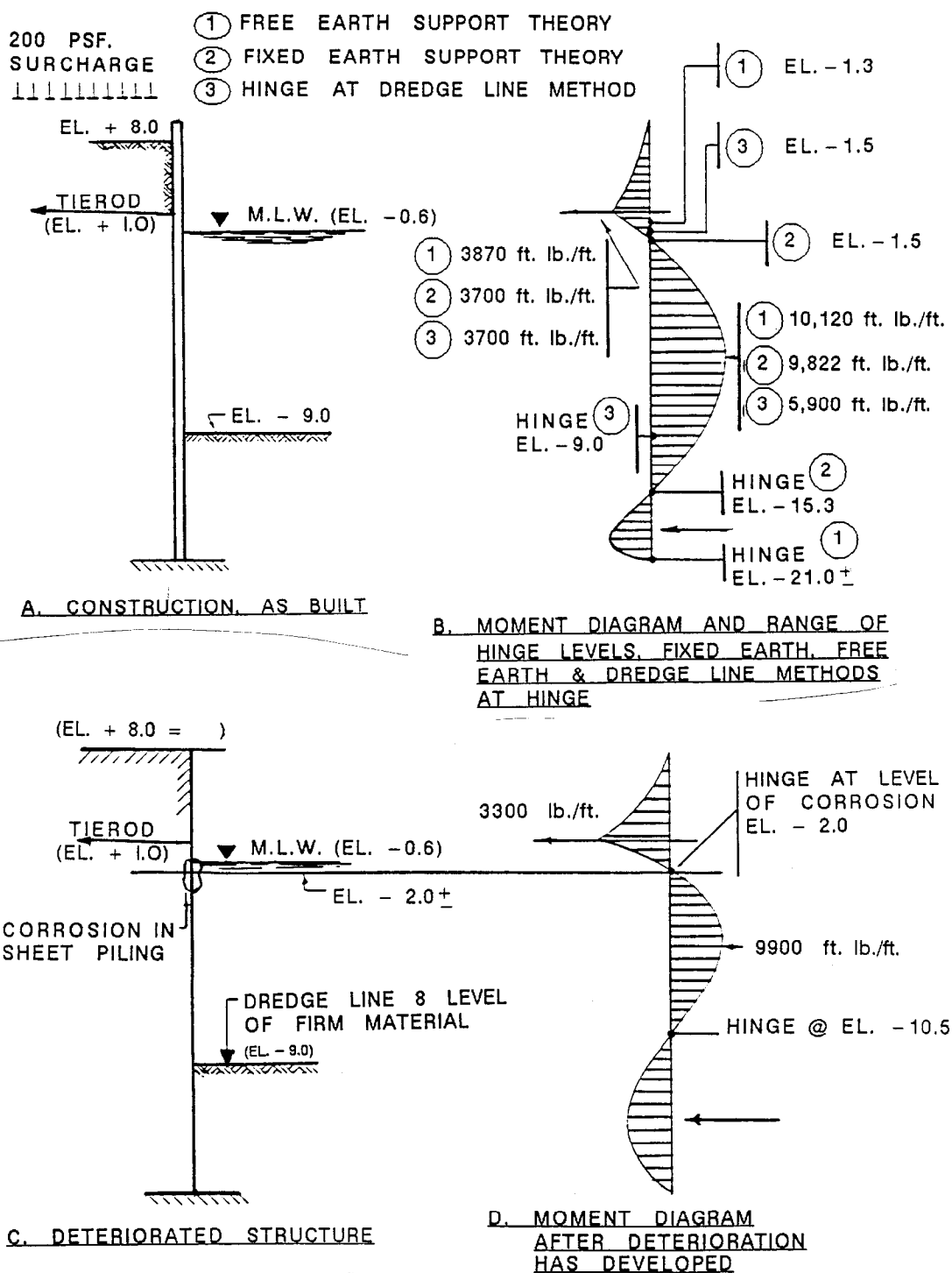
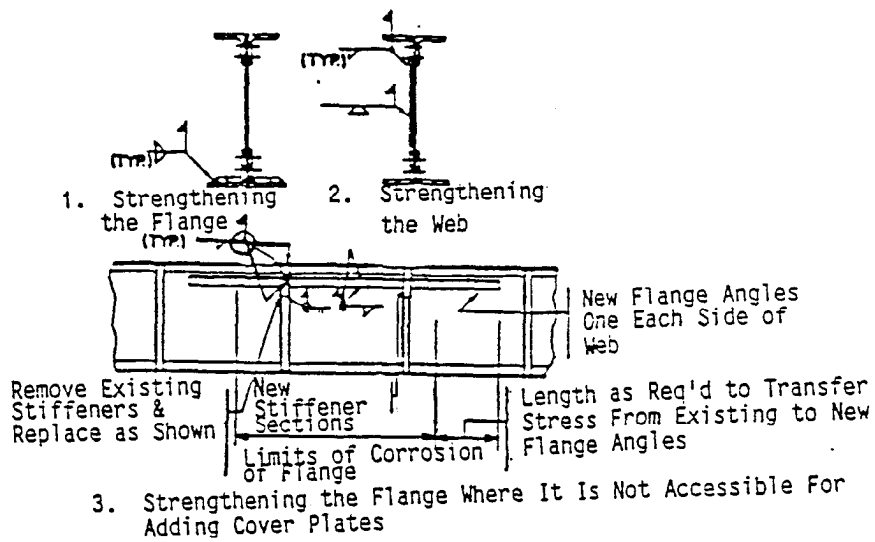
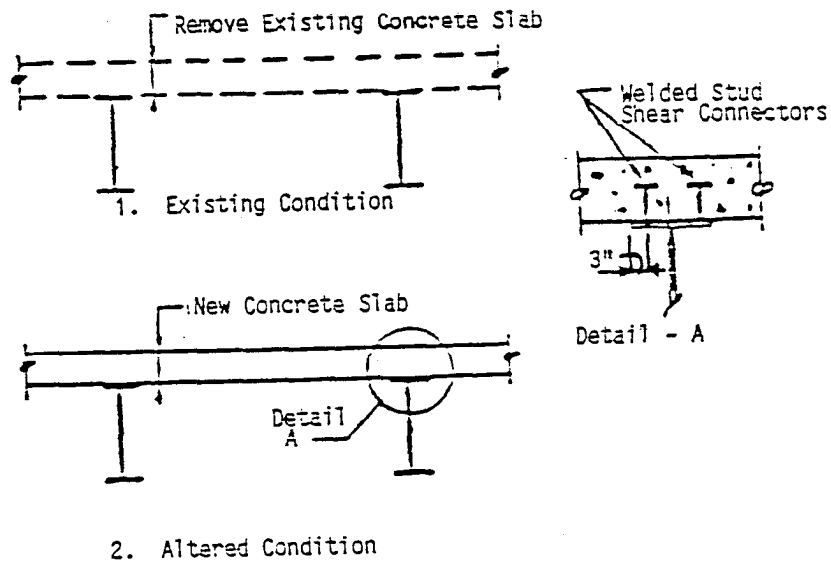


Figure 17  
Change in Flexural Action in Bulkhead  
After Deterioration Has Occurred

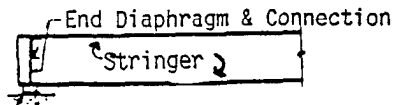
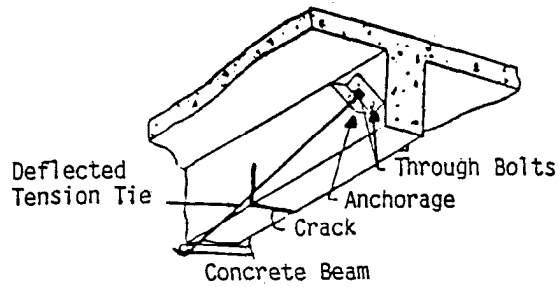


A. PLATING

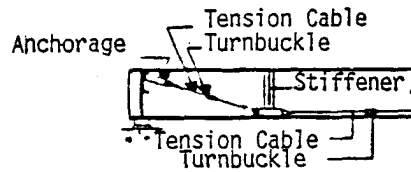


B. INDUCE COMPOSITE ACTION

Figure 18  
Methods of Strengthening an Existing Structure  
Page 1 of 4

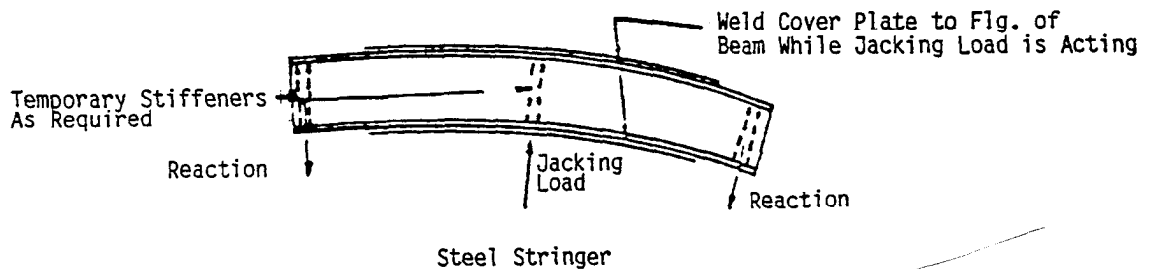


1. Before Strengthening



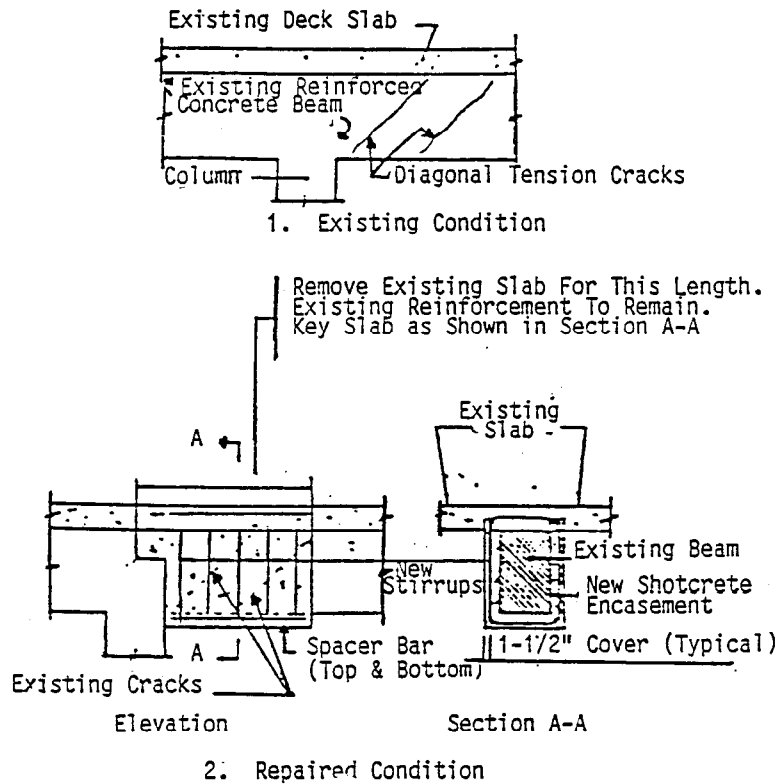
2. After Strengthening

### C. POST STRESSING

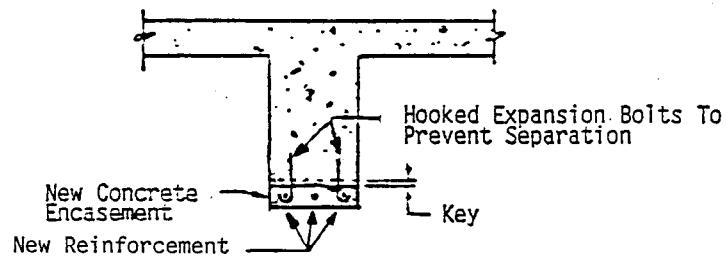


### D. PRESTRESSING

Figure 18  
Methods of Strengthening an Existing Structure  
Page 2 of 4

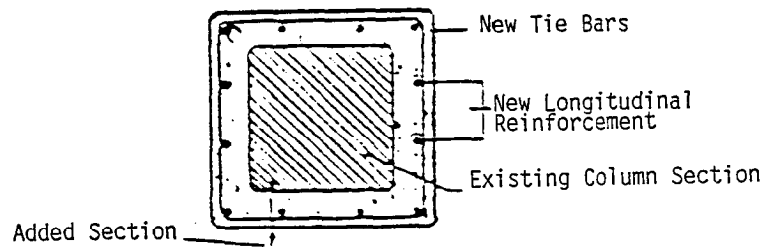
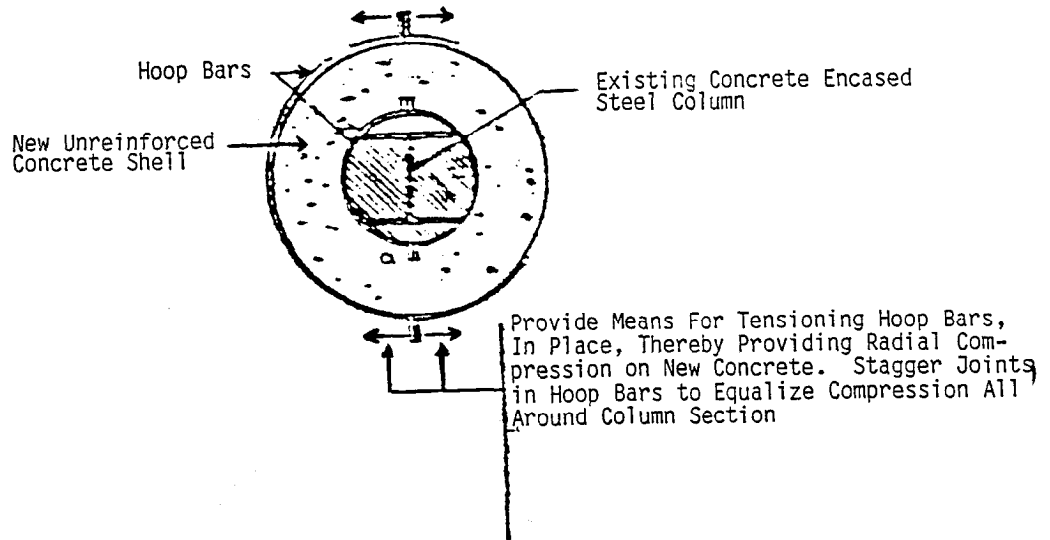


#### E. SHEAR REINFORCEMENT



#### F. FLEXURAL REINFORCEMENT

Figure 18  
Methods of Strengthening an Existing Structure  
Page 3 of 4



#### G. COLUMN REINFORCEMENT

Figure 18  
Methods of Strengthening an Existing Structure  
Page 4 of 4

increased by causing the concrete slab to act compositely with the beam. The slab serves as a top cover plate.

6.5.1.3 Prestressing. Jacks can be used effectively to reduce stresses in existing flanges. Cover plates are welded before removing jacks.

6.5.1.4 Shear Reinforcement. Vertical stirrups serve as hangers that support the beam from the uncracked portion of concrete near the column.

6.5.1.5 Flexural Reinforcement. Longitudinal reinforcement can be added effectively if positive means for preventing separation and for transferring horizontal shear are used.

6.5.1.6 Column Reinforcement. Column sections may be strengthened by adding concrete with longitudinal and lateral reinforcement or by adding unreinforced concrete restrained by hoop bars.

6.5.2 Compatibility. The design details shall encompass any inherent incompatibility of old and new materials. Provision shall be made to resist separation forces. New concrete shall have a different modulus of elasticity, coefficient of thermal expansion, and shrinkage than old concrete. Consider differing expansion effects due to differing absorption of moisture. Provide resistance against "curling" due to thermal gradients.

Compatibility of connectors must be considered. For example, rivets or bolts are not compatible with welds. Friction bolts are not compatible with rivets. Creep is an important factor.

6.5.3 Dead Load Versus Live Load Stresses. Unless the load on a structure is relieved (for example, by removal or by jacking), the existing framing will continue to carry:

- a) the full dead load of the construction,
- b) any part of the live load which is in place when the new framing is connected, and
- c) a proportionate share of the live load subsequently added.

The new framing will carry only a part of the live load. As a result, under the final loading condition, the stresses in the new and existing material of the same or similar members will be different, often radically so.

For example, assuming a 1:1 ratio of dead to live load and of new to existing material in the cross section of a given member and disregarding plastic deformation, the stress in the existing material would be three times the stress in the new. As a result, the new material cannot be stressed up to allowable values without simultaneously overstressing the existing sections. It is necessary either to provide an excess of new material or to relieve the load on the structure before strengthening.

6.5.4 Exception. Para. 6.5.3 may not apply if plastic deformation of the structure (and its associated, increased deflection) can be permitted. For example, as regards flexural members, in general, the plastic hinge moment

capacity is not reduced by locked-in stresses, but for compression members, locked-in stresses reduce the buckling strength. The magnitude of the reduction is a function of the  $L/r$  value shown in Figure 19.

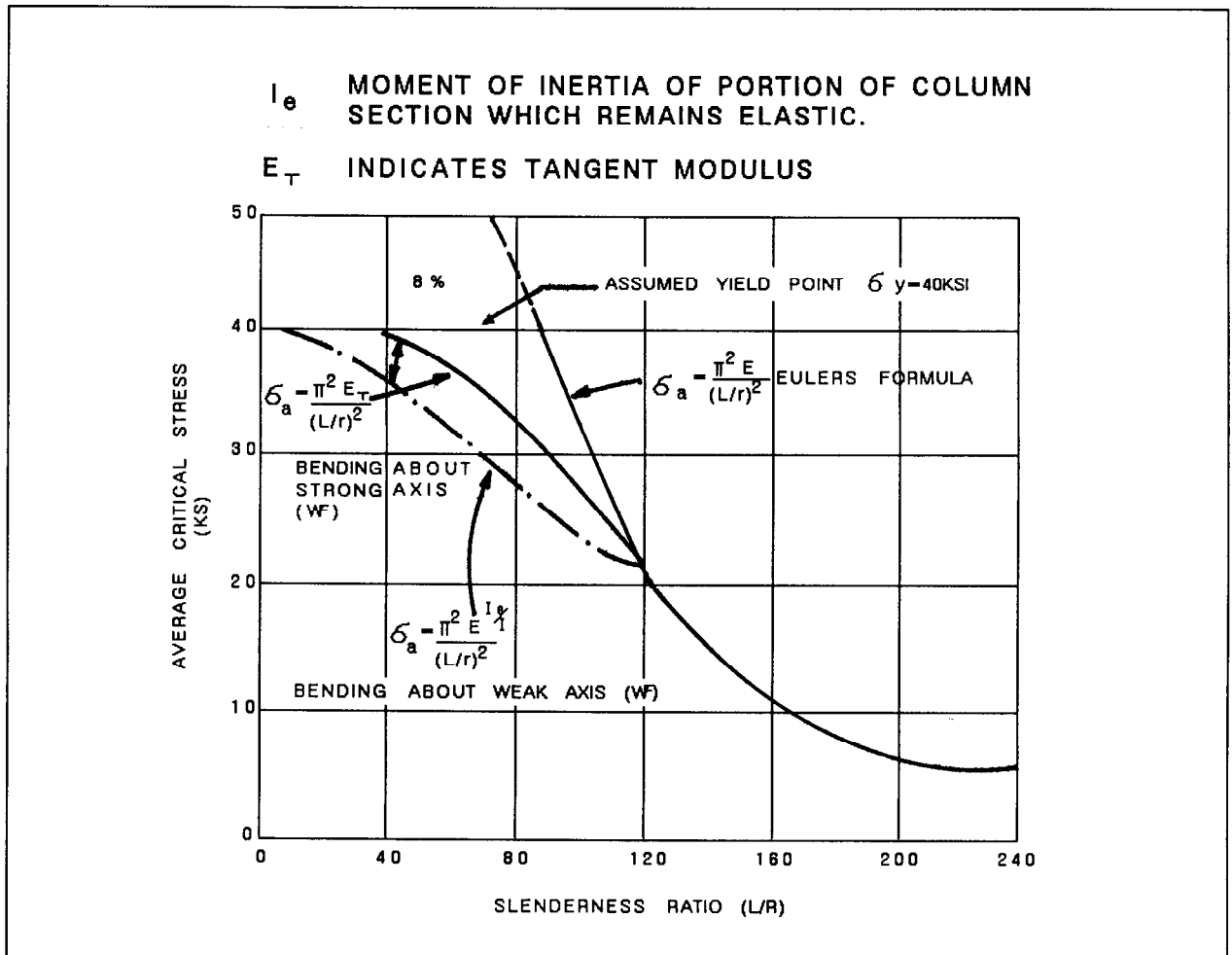


Figure 19  
Effects of Residual Stress on Column Strength

## Section 7: DETERIORATION OF WATERFRONT STRUCTURES

7.1 Causes. The more common causes of deterioration associated with steel, concrete, and timber waterfront structures are given in paras. 7.1.1 through 7.1.3.

7.1.1 Steel Structures. For steel structures deterioration is caused by:

- a) corrosion (see Figures 20 through 22),
- b) abrasion (see Figure 23), and
- c) impact.

7.1.2 Concrete Structures. In concrete structures deterioration is caused by:

- a) corrosion of reinforcement,
- b) chemical reactions (see Figure 24),
- c) weathering (see Figure 25),
- d) swelling of concrete (see Figure 26), and
- e) impact (see Figure 27).

7.1.3 Timber Structures. In timber structures deterioration is caused by:

- a) corrosion and abrasion of hardware (see Figures 28 and 29),
- b) borer attack,
- c) decay, and
- d) impact.

7.2 Preventive Measures in Design and Construction

7.2.1 Steel Structures

7.2.1.1 General. All parts that will be subject to corrosion shall be accessible for inspection and repair. If not accessible, encase with concrete or provide some other long-life, high-resistance type of coating.

7.2.1.2 Shapes. Shapes shall be selected that have a minimum of exposed surface. For example, use T's instead of double angles.

7.2.1.3 Detailing. Detailing shall be designed so that accumulations of dirt and debris will be avoided. Avoid narrow crevices that cannot be painted or sealed. Draw faying surfaces into tight contact by use of closely spaced stitch rivets, bolts, or welds. Prime faying surfaces before assembly.

7.2.1.4 Minimum Thickness of Metal. For piling, deck and substructure framing and bracing, and hardware and fittings, refer to Sections 2, 3, and 4 respectively. For other framing, no minimum thickness requirement is established.

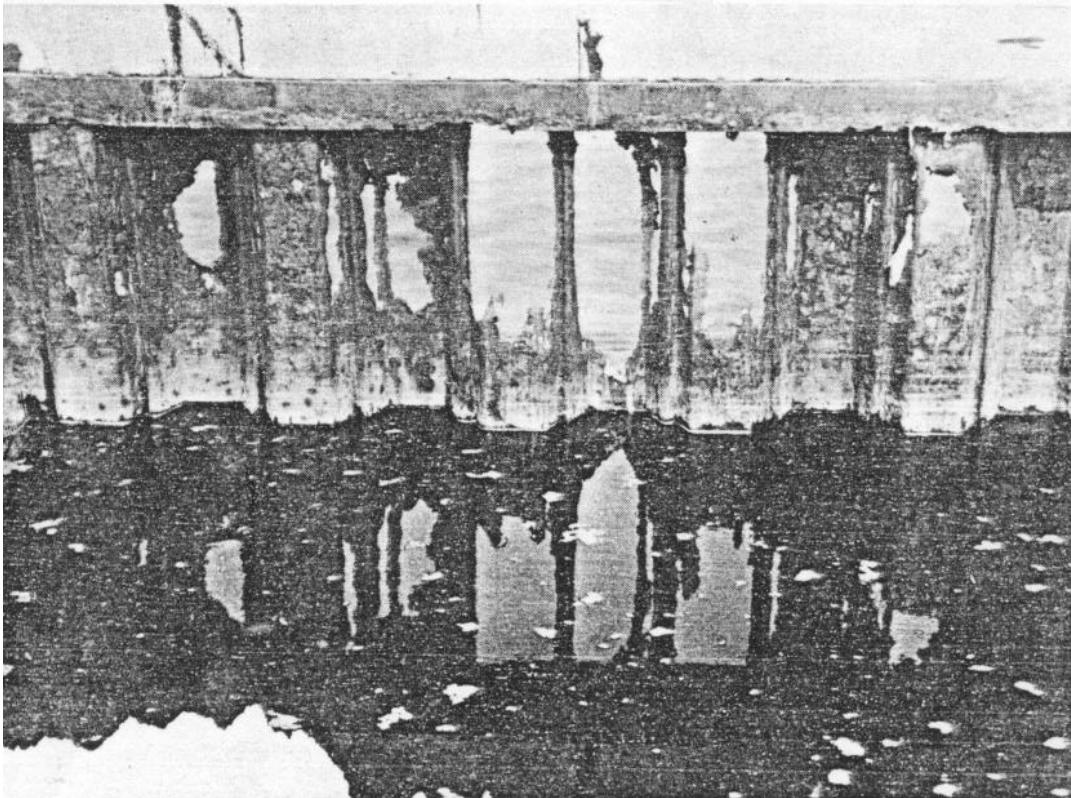


Figure 20  
An Advanced Case of  
Corrosion of Steel Sheet Piling



21(a)



21(b)

NOTE : Figure 21(a) shows the appearance of the beam before removal of the rust scale. Except for some pitting and tuberculations, it looks relatively sound. Figure 21(b) shows the same view after the rust scale had been partly removed by hammering.

Note that the entire appearance has changed, revealing a very serious condition of corrosion. The flange has been reduced to sheet metal, with holes indicated by the arrows.

These photographs illustrate the fact that the severity of corrosion in a steel member should not be judged until after the rust scale has been removed.

Figure 21  
Corrosion of Low Water Brace Beam



Figure 22  
Corrosion of Steel H-Piles

7.2.1.5 Drainage. In general, detailing framing to shed water is the single most important factor in inhibiting corrosion and deterioration of coatings (except the need for good workmanship). If the potential for ponding is unavoidable, provide drain holes. Drain holes shall be a minimum of 4 in. (101.6 mm) in diameter to inhibit clogging.

7.2.1.6 Sacrificial Metal. The use of sacrificial metal shall be avoided in favor of using protective coatings.

7.2.1.7 Cathodic Protection. For information pertaining to Cathodic protection refer to **para. 5.3**.

## 7.2.2 Concrete Structures

7.2.2.1 Class of Concrete. For information pertaining to concrete classes refer to Sections 2 and 3.

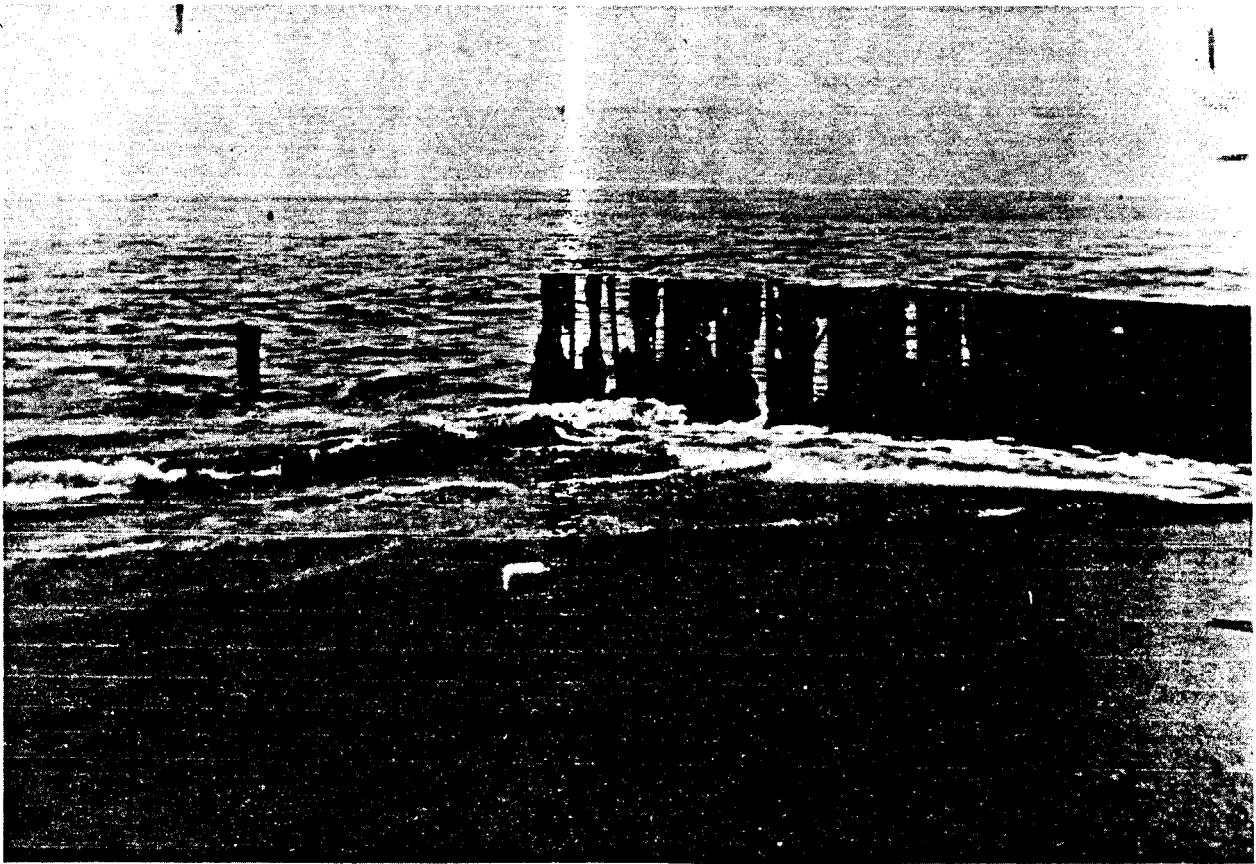
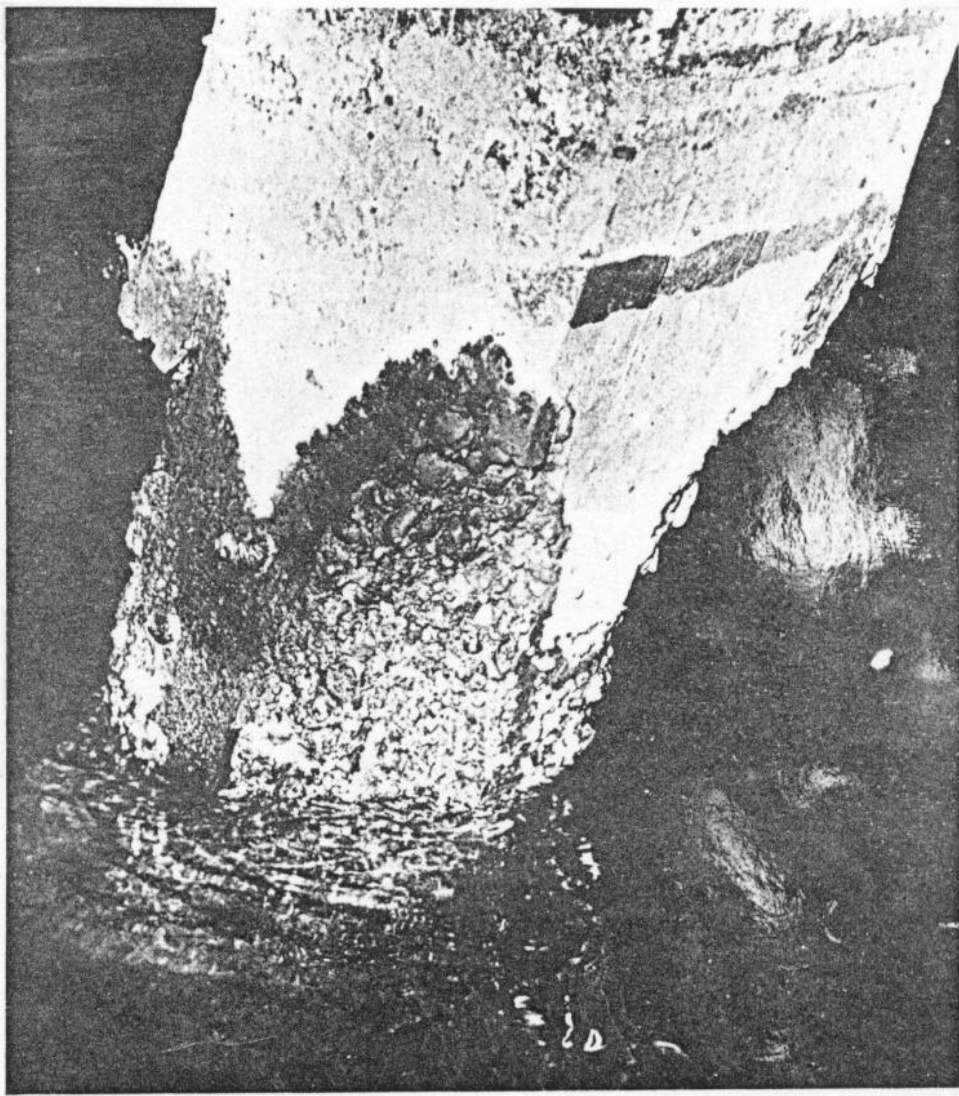


Figure 23  
Result of Abrasion of  
Steel Sheet Piling in Surface Zone

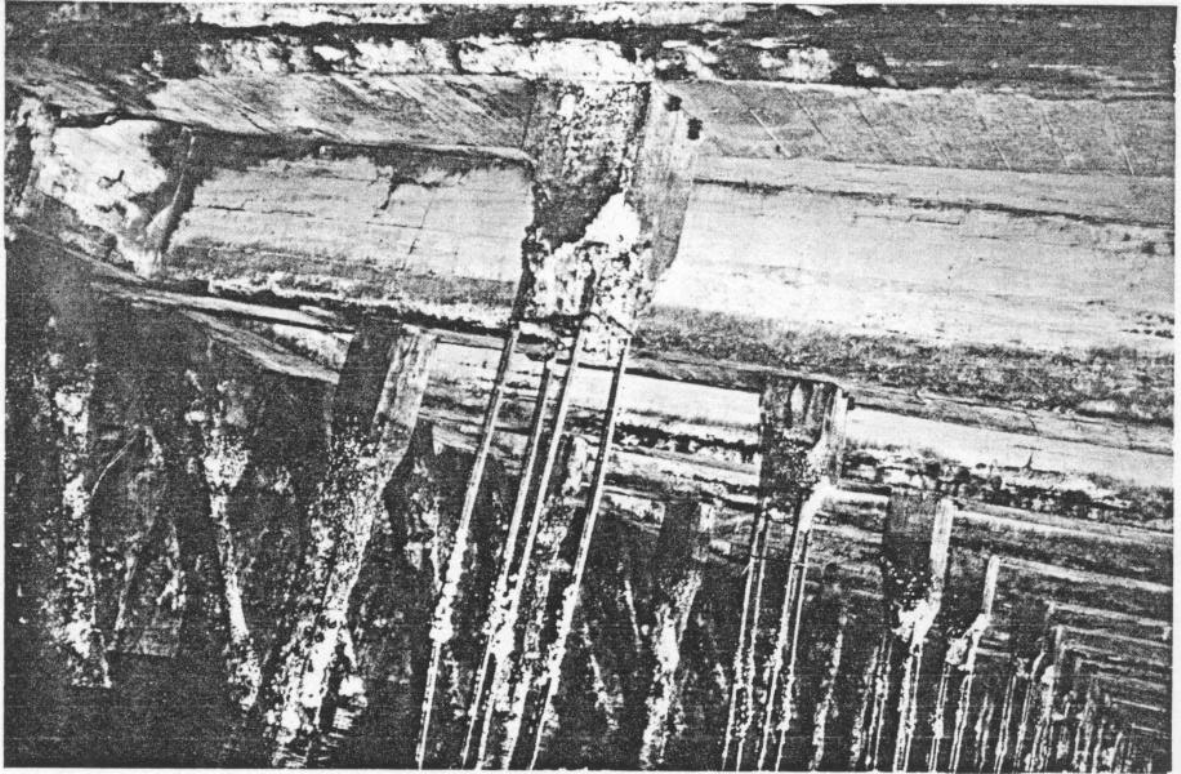
7.2.2.2 Cover Over Reinforcement. For information pertaining to covering for reinforcement, refer to Sections 2 and 3.

7.2.2.3 Quality of Concrete. Good quality is the important factor in obtaining a dense concrete. This, in turn, is the most important factor in preventing penetration of moisture, which is the primary cause of deterioration of concrete. Do not use poorly graded aggregate, or a water-cement ratio greater than 6 gal (22.71 **l**)/**sack** of cement, reduced to 5 gal (18.92 **l**)/**sack** of cement for thinner sections such as slabs and wherever clear cover over reinforcement is 2 in. (50.8 mm) or less. Watertight concrete can be obtained by using air entrainment (maximum 6 percent by volume) and a water-cement ratio not greater than 5 gal/sack of cement.



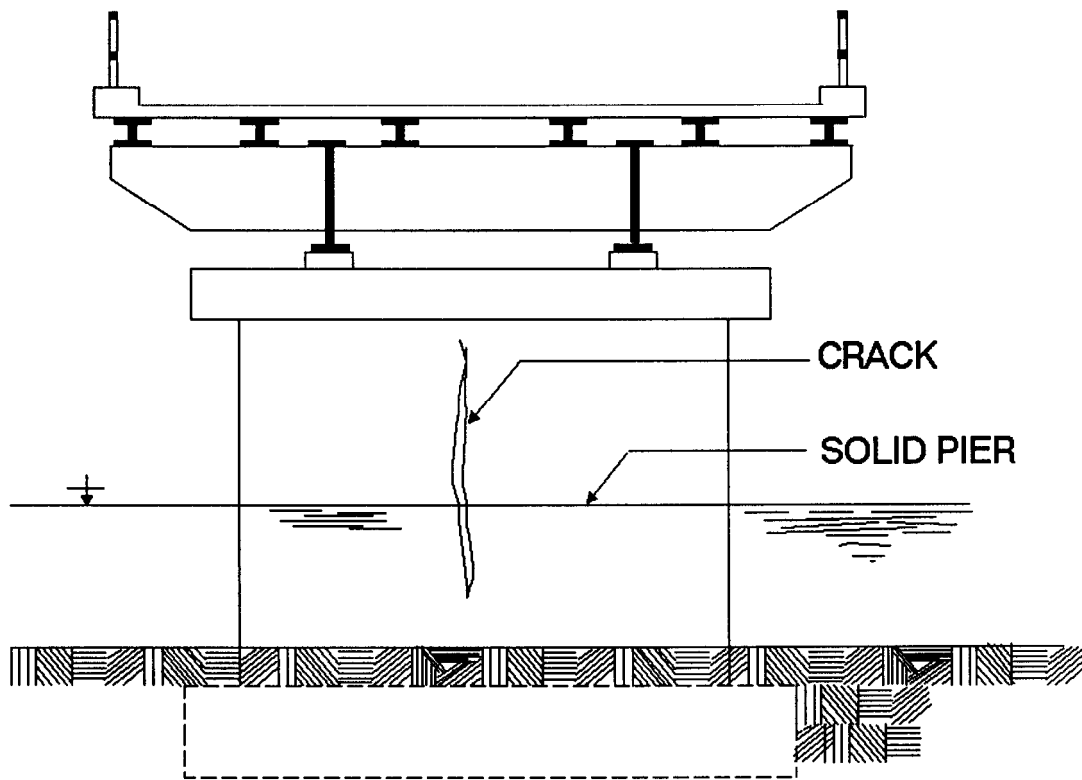
Notice the protrusion of aggregate particles from cement-sand matrix; abrupt limit of deterioration near high water mark (black horizontal band on the pile); and spalling of corners due to corrosion of corner bars.

Figure 24  
View of Concrete Pile  
Deteriorated by Seawater--Sulphate Attack



Deterioration of precast concrete piling in a marine environment. Observe the condition of the piles in the rear row of verticals. The condition shown was attributed to weathering plus abrasion by floating ice. The reinforcing bars are still in relatively good condition. The structure was about 17 years old when this photograph was taken.

Figure 25  
Spalling of Concrete Due to Weathering  
(Freeze-Thaw)



This pier was built in a cofferdam. Concrete was placed "in-the-dry." After removal of the cofferdam, the dry concrete became saturated and swelled. Above the saturation line the concrete did not swell. Cracking resulted from the differential expansion. Heavy reinforcement in the cap section prevented the crack from propagating to the top of the pier.

Figure 26  
Swelling of Concrete Due to Absorption of Water

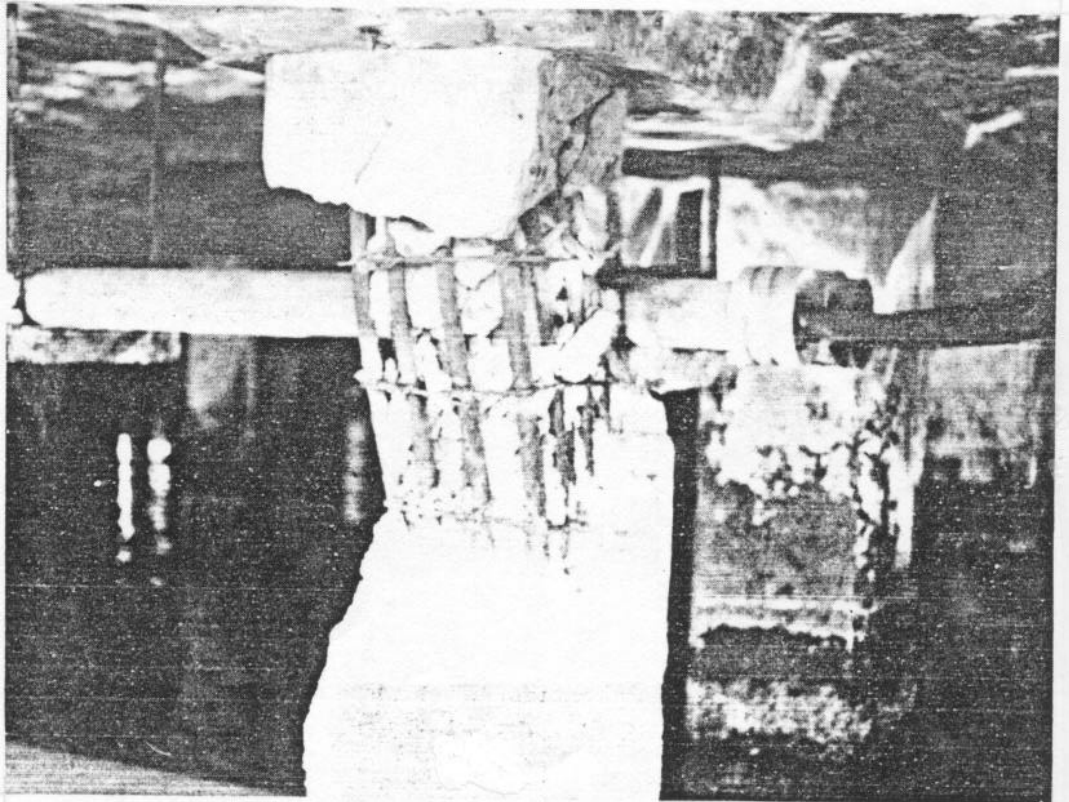
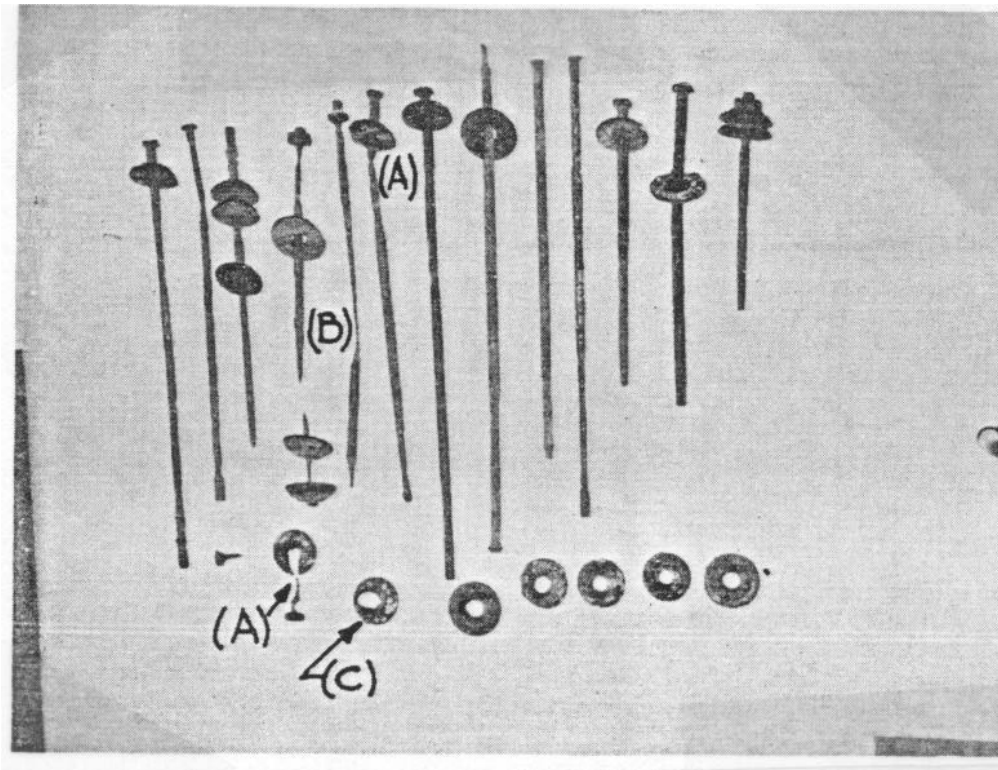


Figure 27  
Damage to Concrete Pile Due to Berthing Impact



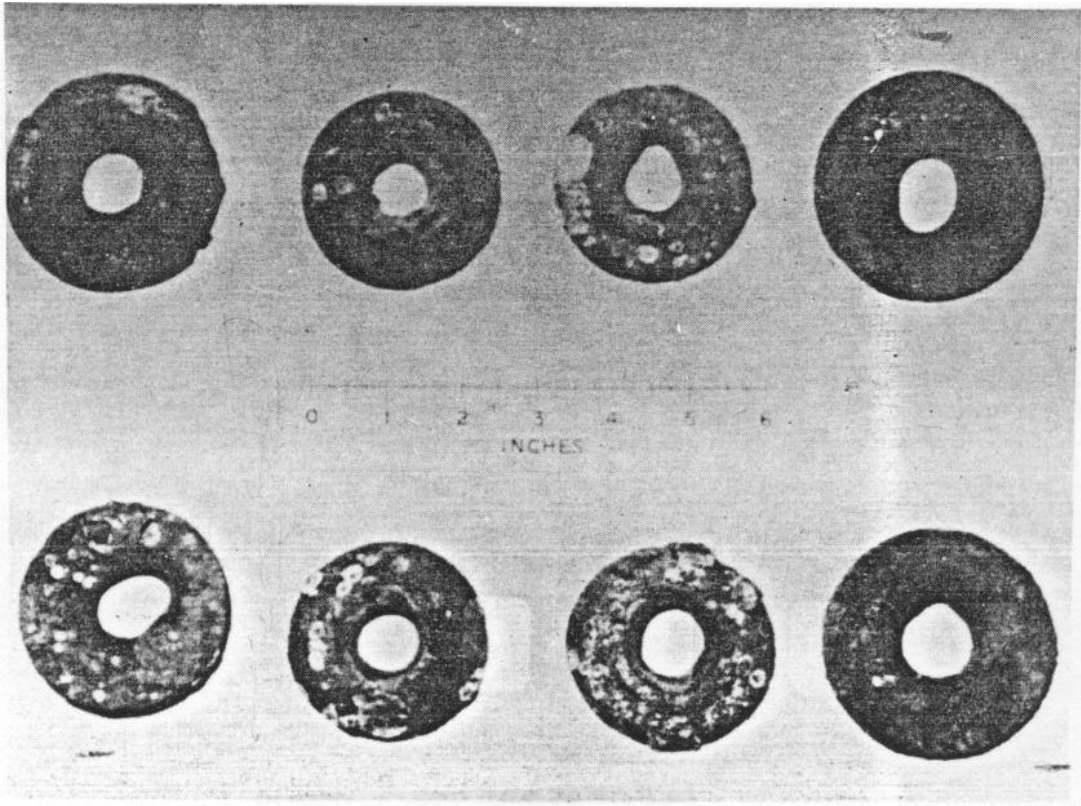
Notice the necking-down of bolts and that the necking-down is localized where the bolt shank was exposed (next to the washers (A), and between adjacent timbers (B)). Note also, the distortion and enlargement of the holes in the washers (C). Observe also that some of the bolts have no heads; they were corroded away. Similarly, no nuts were found; presumably, they too were corroded away. The bolts were held in place by friction until lateral forces due to waves and ice pulled them loose, at which point bolts and timbers were lost.

Figure 28  
Samples of Hardware Taken From Pier Structure

**7.2.2.4 Types of Concrete.** Type III (high early strength) cement is excessively susceptible to sulphate attack, and shall not be used. In general, avoid the use of Type I cement in a saltwater environment. Type II (sulphate-resistant) cement shall be used. The use of Type V (high sulphate-resistant) cement is seldom required.

**7.2.2.5 Expansion Joints.** Provision shall be made for an adequate number of expansion joints. Use types of expansion joints such as double bents with movement taken up by bending of the piles or elastomeric pads (see Figure 30) with some form of joint sealer. Types of expansion joints to be avoided are shown in Figure 31. Experience indicates that such joints do not give good service.

**7.2.2.6 Tropical Climates.** In tropical climates, in areas subject to salt spray, consider the use of galvanized or plastic coated reinforcing bars. If plastic coated bars are used, pay special attention to bond stresses.



Notice the distortion and enlargement of holes in washers. Corrosion of bolt heads and nuts loosened the connections. The resulting working of the timbers caused the bolt shank to "saw" on the washers, wearing both shank and washer. After a while, the hole in the washer was enlarged and the size of the bolt head was reduced (by corrosion), permitting the bolt head to be drawn through the hole in the washer, thus freeing the connection.

Figure 29  
Detail of Washers Shown in Figure 28

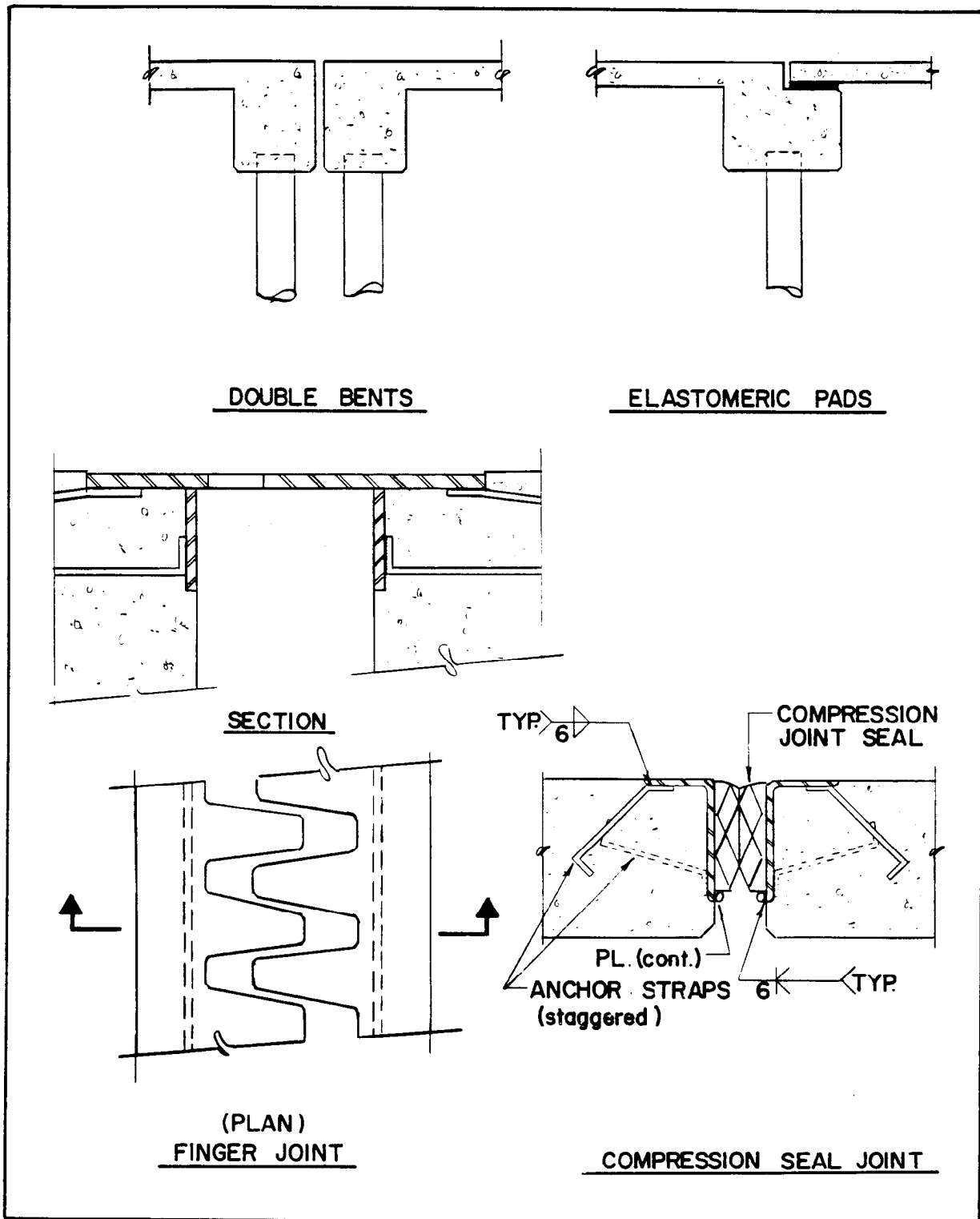
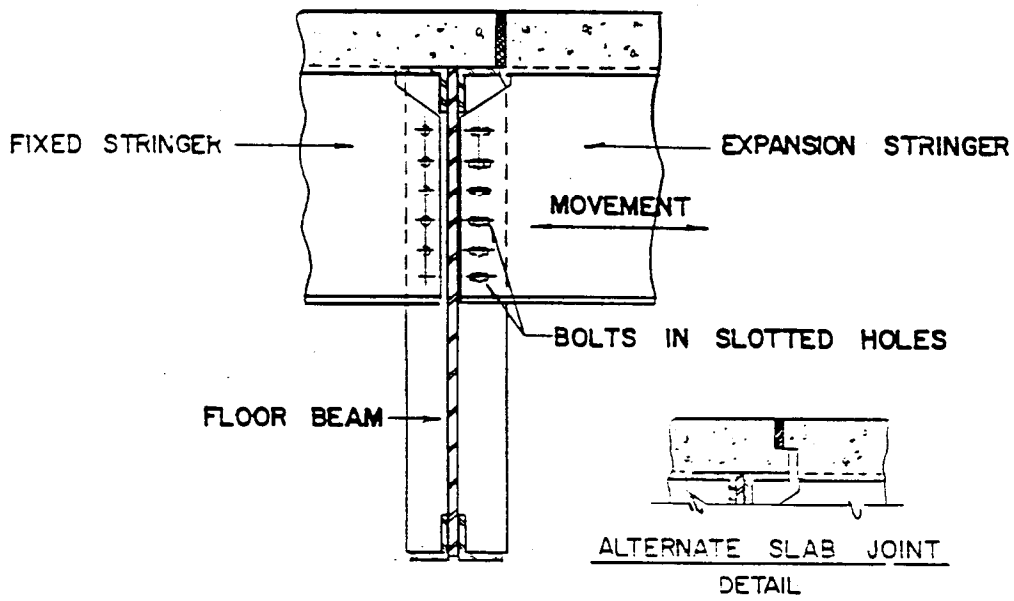
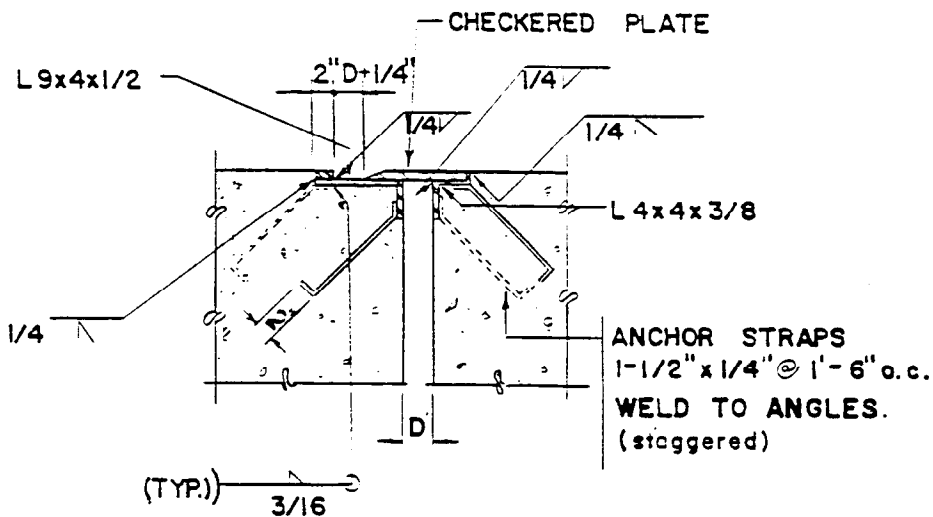


Figure 30  
Recommended Types of Expansion Joints



### SLOTTED HOLE EXPANSION ASSEMBLY



### EXPANSION PLATE ASSEMBLY

Figure 31  
Types of Expansion Joints Not Recommended

7.2.2.7 Mix. Excessively rich mixes, over 6 bags per  $\text{yd}^3$  ( $0.764 \text{ m}^3$ ), shall be avoided, as excess cement tends to enhance the potential chemical reaction with seawater.

7.2.2.8 Jackets and Facings. Timber jackets for concrete piles (see Figure 32) and stone facing for concrete seawalls work extremely well to prevent deterioration due to corrosion of reinforcement, weathering, and chemical attack. They tend to isolate the concrete from chemical constituents in the environment, insulate against freezing, and keep free oxygen from the reinforcing bars. See Figure 26 for comparison of precast concrete piling installed without timber jackets.

7.2.2.9 Aggregate. For most aggregates, alkali-aggregate reaction can be prevented by specifying maximum alkali content of the cement (percent  $\text{Na}_2\text{O}$ , plus 0.658 times percent  $\text{K}_2\text{O}$ ) not to exceed 0.60 percent.

7.2.2.10 Surf Zone. In a surf zone, the concrete cover and streamline sections shall be increased to prevent abrasion. The best solution, if economically feasible, is a granite or other hard facing.

7.2.2.11 Additives. Calcium chloride (as an accelerator) shall not be used in prestressed concrete and concrete exposed to seawater. The use of calcium nitrite or other chemicals as a deterrent to corrosion of embedded reinforcing steel is not an adequate substitute for good quality concrete and adequate cover. In extreme cases, use of coated reinforcing bars may be required.

7.2.2.12 Drainage. Where feasible, scuppers and weep holes shall be detailed to drip clear of the underlying structure. Provide drip grooves in fascia beams and slab soffits.

7.2.2.13 Compatibility of Sections and Materials. See Figures 33 and 34 for some common problems.

7.2.3 Timber Structures. Timber structures shall conform to the following criteria:

a) Design detail shall minimize cutting, especially that which must be done after treatment.

b) Design detail shall provide for ventilation around timbers. Avoid multiple layers of timbers as decay is enhanced by moist conditions at facing surfaces. Curb logs shall be set up on blocks. Walers shall be blocked out from face of pier. Thin spacers between chocks and wales, and gaps between deck and tread planks shall be provided.

c) For requirements concerning hardware, preservative treatment of the wood, protection of tops of piles, and selection of species of wood, refer to Sections 2 and 3.

## 7.3 Case Histories

### 7.3.1 Case History No. 1--Corrosion and Berthing Impact

7.3.1.1 The Problem. Existing sheet pile wall damaged by corrosion and berthing impacts.

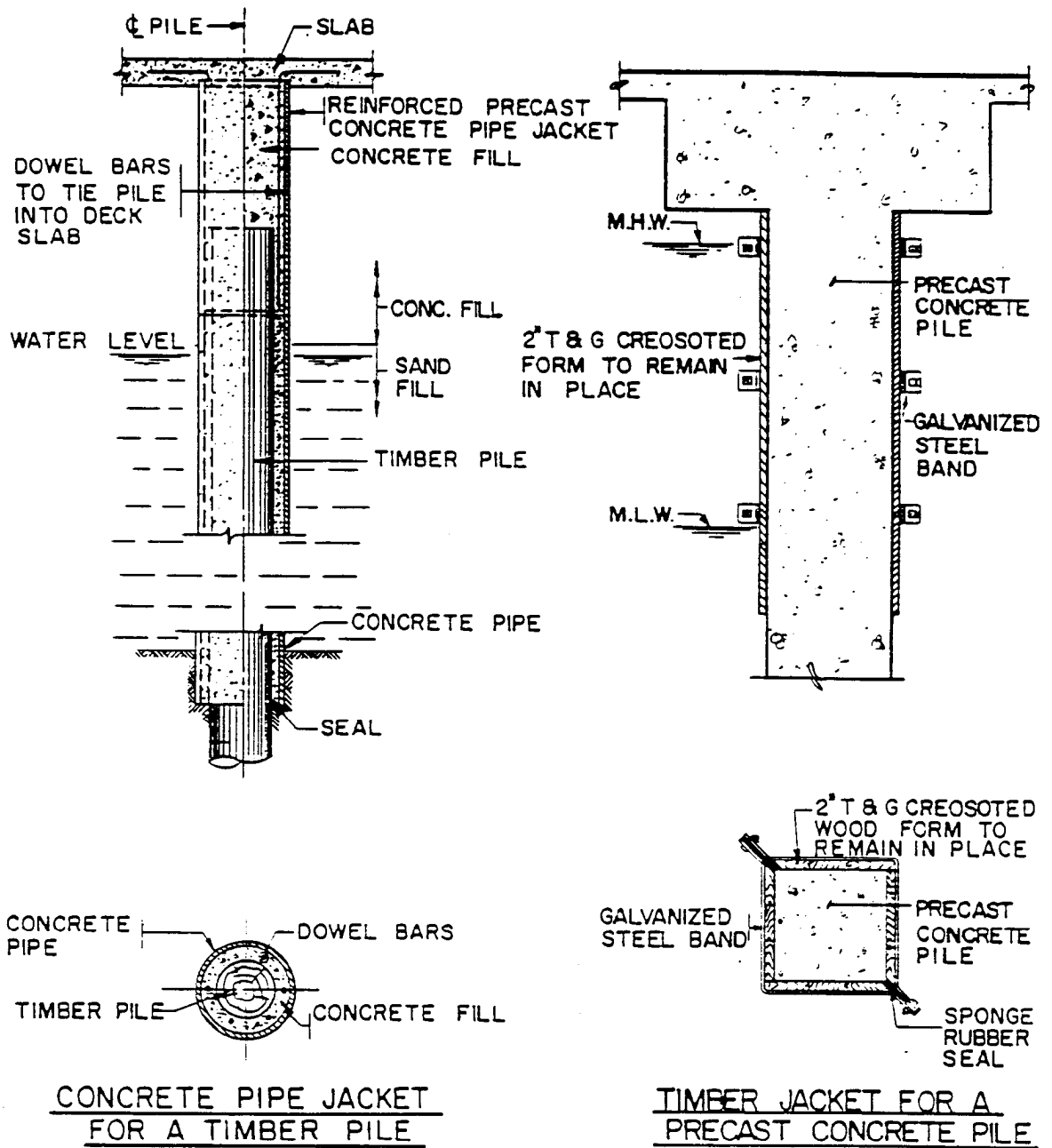
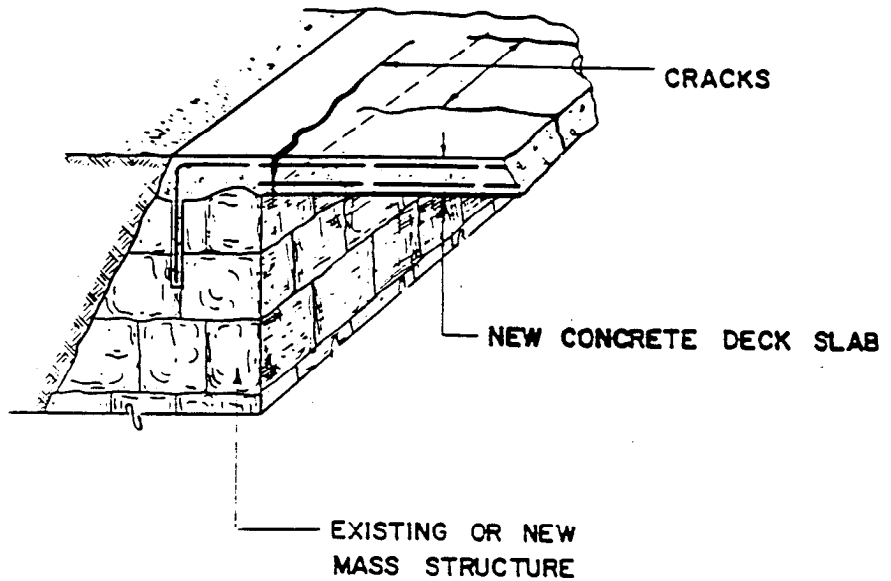
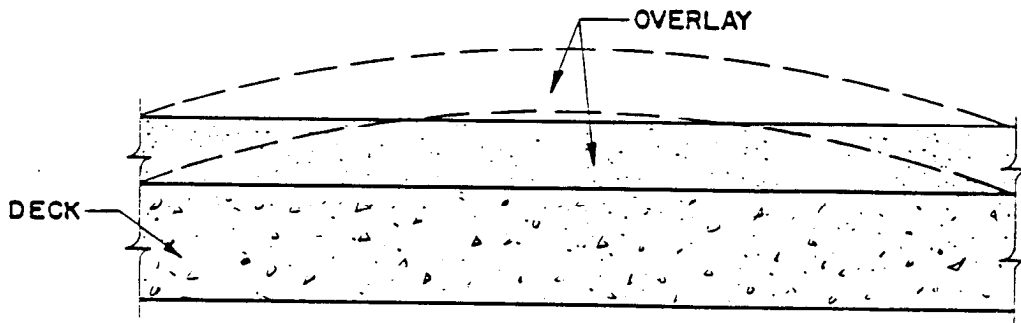


Figure 32  
Pile Jackets

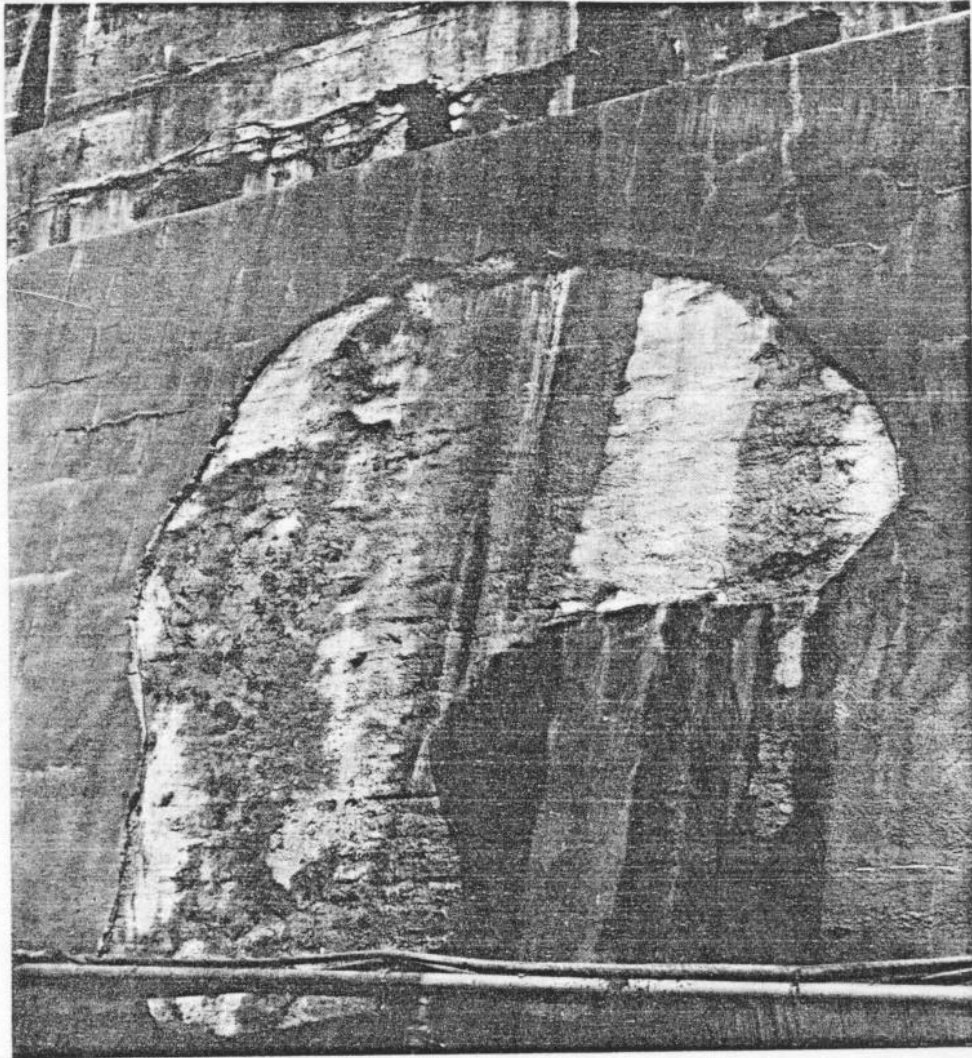


CRACKING DUE TO AN ABRUPT CHANGE IN SECTION OF  
A CONCRETE STRUCTURE



BUCKLING OF APPLIED OVERLAY DUE TO A DIFFERENCE  
IN MATERIAL PROPERTIES

Figure 33  
Compatibility of Sections and Materials



Note: Proper detailing would provide expansion anchors and mesh reinforcement.

Figure 34  
Spalling of Shotcrete Repair  
Due to Lack of Anchorage to Base Matrix

7.3.1.2 The Solution. A new bulkhead wall was built (see Figure 35) with existing wall acting as anchorage (bin structure). Construction was confined to the area outboard of existing wall to avoid disturbing a maze of existing utilities.

After about 15 years of service, failures began to occur due to the U-bolts (Plan Section A-A of Figure 35) tearing the interlocks to which they were anchored. Up to 6 in. (152.4 mm) of deflection of the top of the outboard sheet piles had developed. The condition was then repaired (see Figure 36). Figure 37 shows the final construction.

### 7.3.2 Case History No. 2--Corrosion of Sheet-Pile Wall

7.3.2.1 The Problem. An existing sheet pile wall severely damaged by corrosion (see Figure 38).

7.3.2.2 The Solution. The existing sheet piling was "preserved" by external facing (see Figure 39). Corrosion loss due to attack from inside face was assumed to be negligibly small. The strength of the existing wall was checked as indicated in Figure 17.

### 7.3.3 Case History No. 3--Sulfate Corrosion of Concrete Seawall

7.3.3.1 The Problem. Sulfate attack caused disintegration of concrete seawall.

7.3.3.2 The Solution. The contact between existing concrete and seawater (reacting ingredients) was cut off by interposing a layer of sulfate-resistant concrete (see Figure 40). Note that all of the old concrete was not replaced -- only what had been severely attacked. Concept is to inhibit further attack by isolating concrete from aggressive environment.

### 7.3.4 Case History No. 4--Corrosion of Steel H-Piles (Treated)

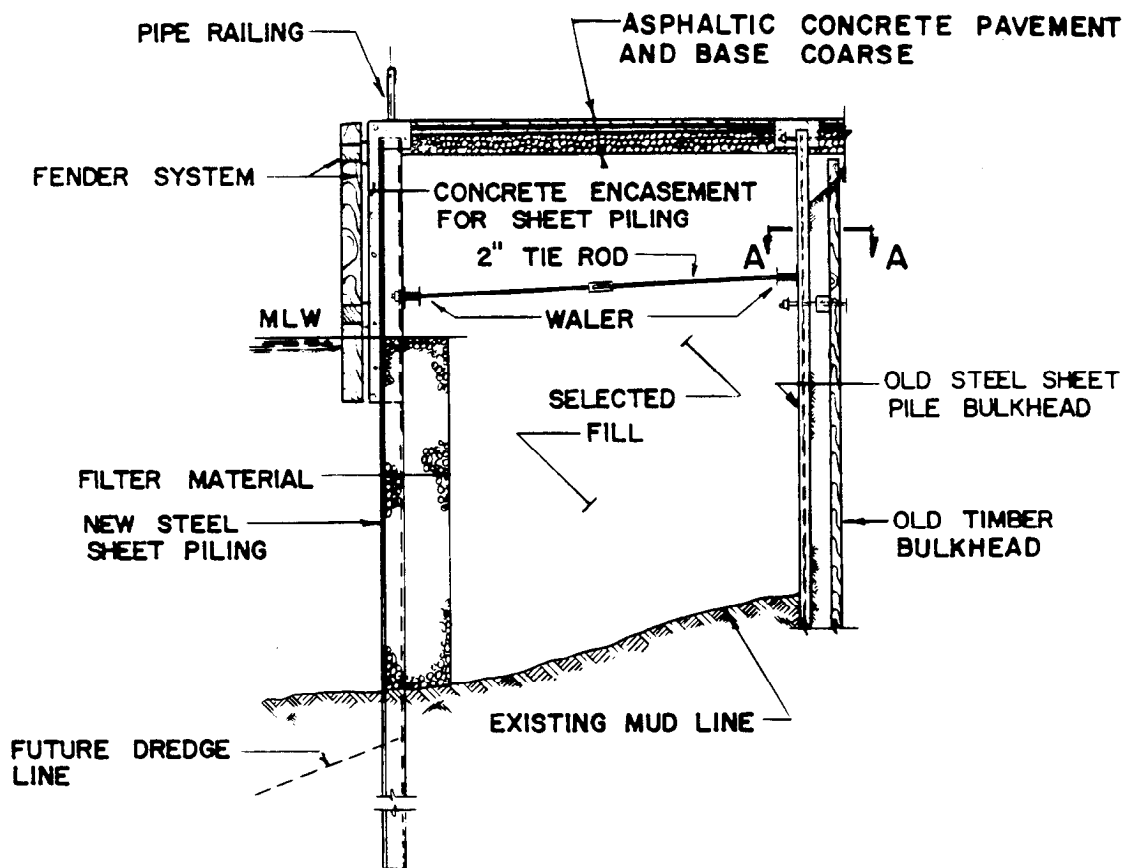
7.3.4.1 The Problem. Corrosion of steel H-piles (see Figure 22).

7.3.4.2 The Solution. Corrosion was concentrated in a narrow band near and below mean low water (see Figure 41). This is a typical condition. Above and below this band, sufficient section remained in piling that, projecting an estimated rate of loss based on observed rate of loss, 25 years of service life remained before critical stress levels (defined as load factor of 1.33 on dead plus live load) would be reached.

### 7.3.5 Case History No. 5--Rotting of Wood Piles

7.3.5.1 The Problem. Wood pier deteriorated due to rot (see Figure 42 for typical pile condition). Below midtide level piles were sound. Deck timbers were creosoted and piles were untreated. There was no borer activity, due to polluted harbor waters.

7.3.5.2 The Solution. The superstructure and deck were removed, and existing piles posted above midtide level (see Figure 43), and a new deck built on posted piles. Salvage of lower portions of existing piles (and of some of the cross caps) reduced repair cost by an estimated 30 to 40 percent.



### SECTION OF NEW BIN CONSTRUCTION

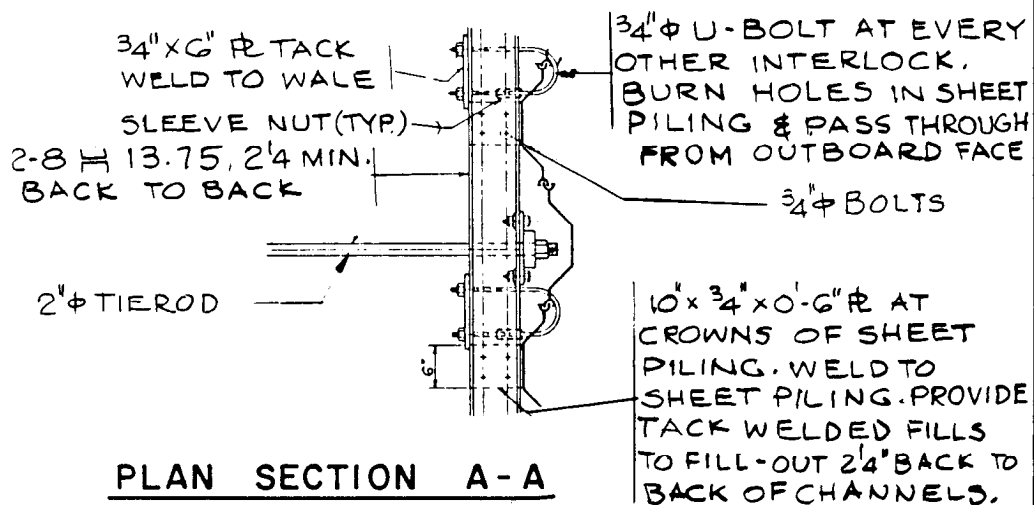


Figure 35  
Case History No. 1--First Repair

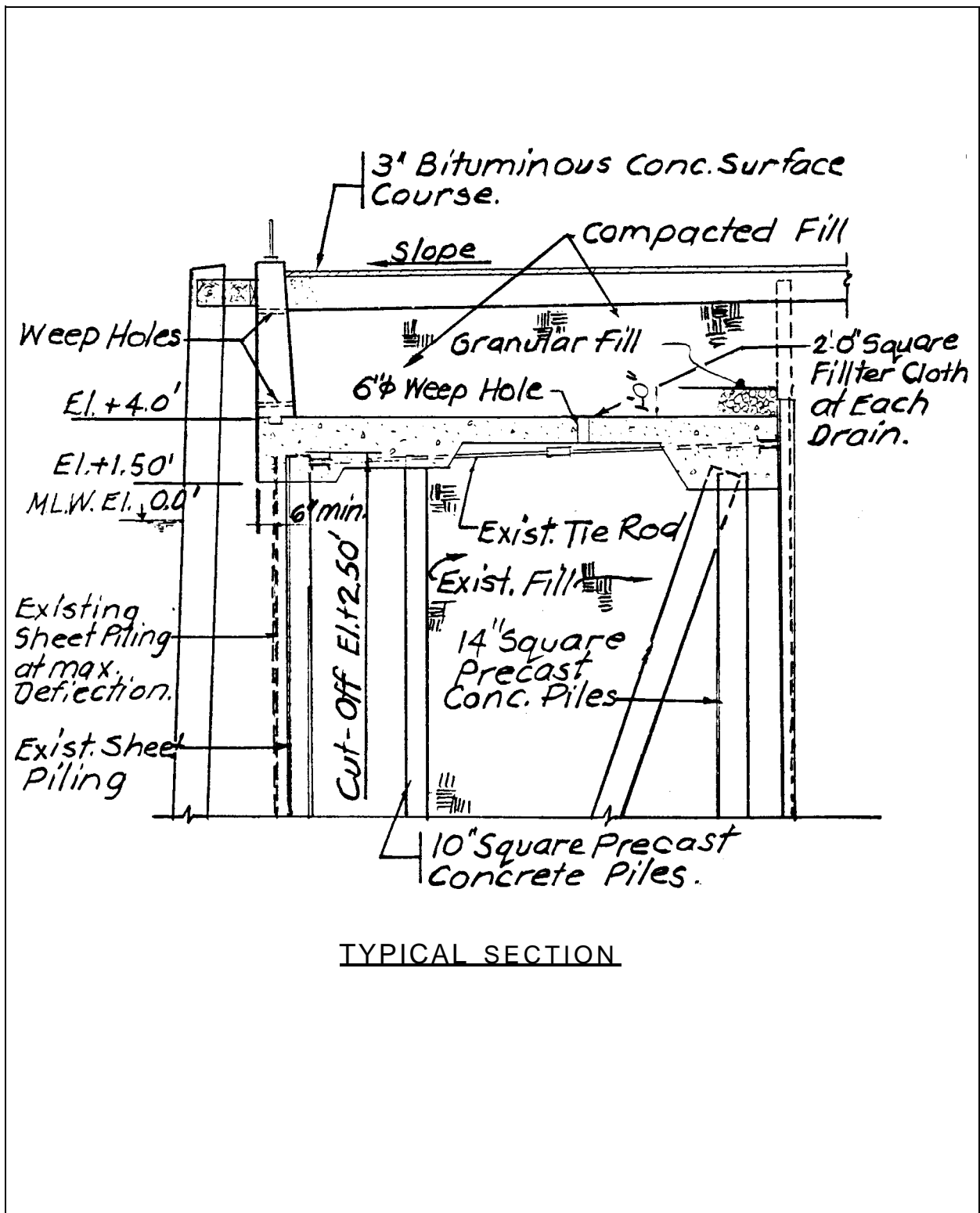


Figure 36  
Case History No. 1--Second Repair



Figure 37  
Case History No. 1--Final Construction

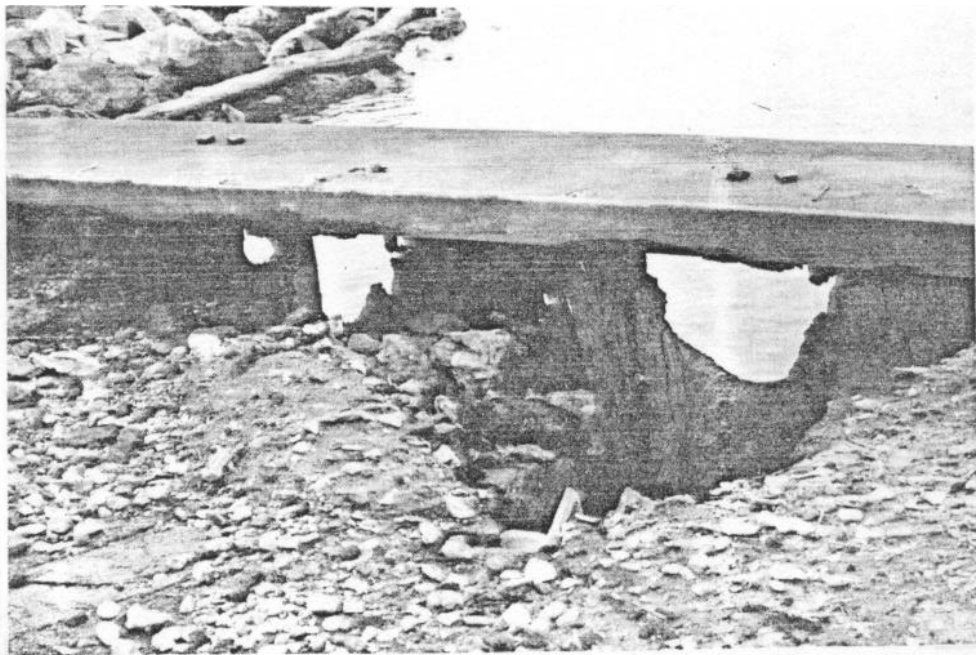
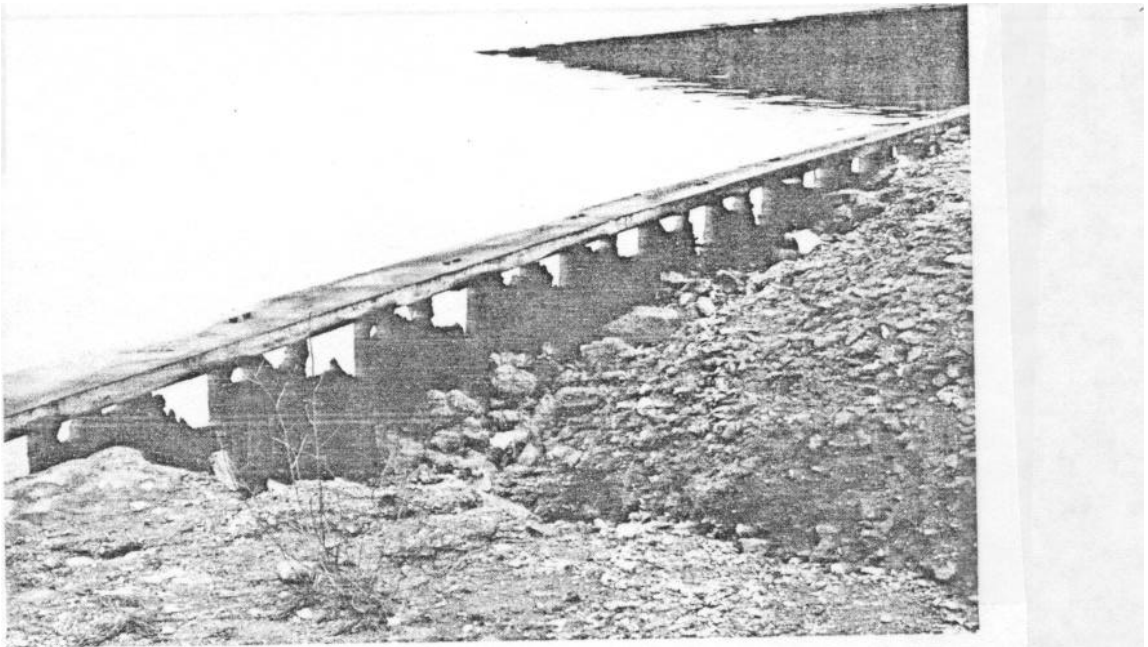


Figure 38  
Case History No. 2--Deterioration of Top of Bulkhead

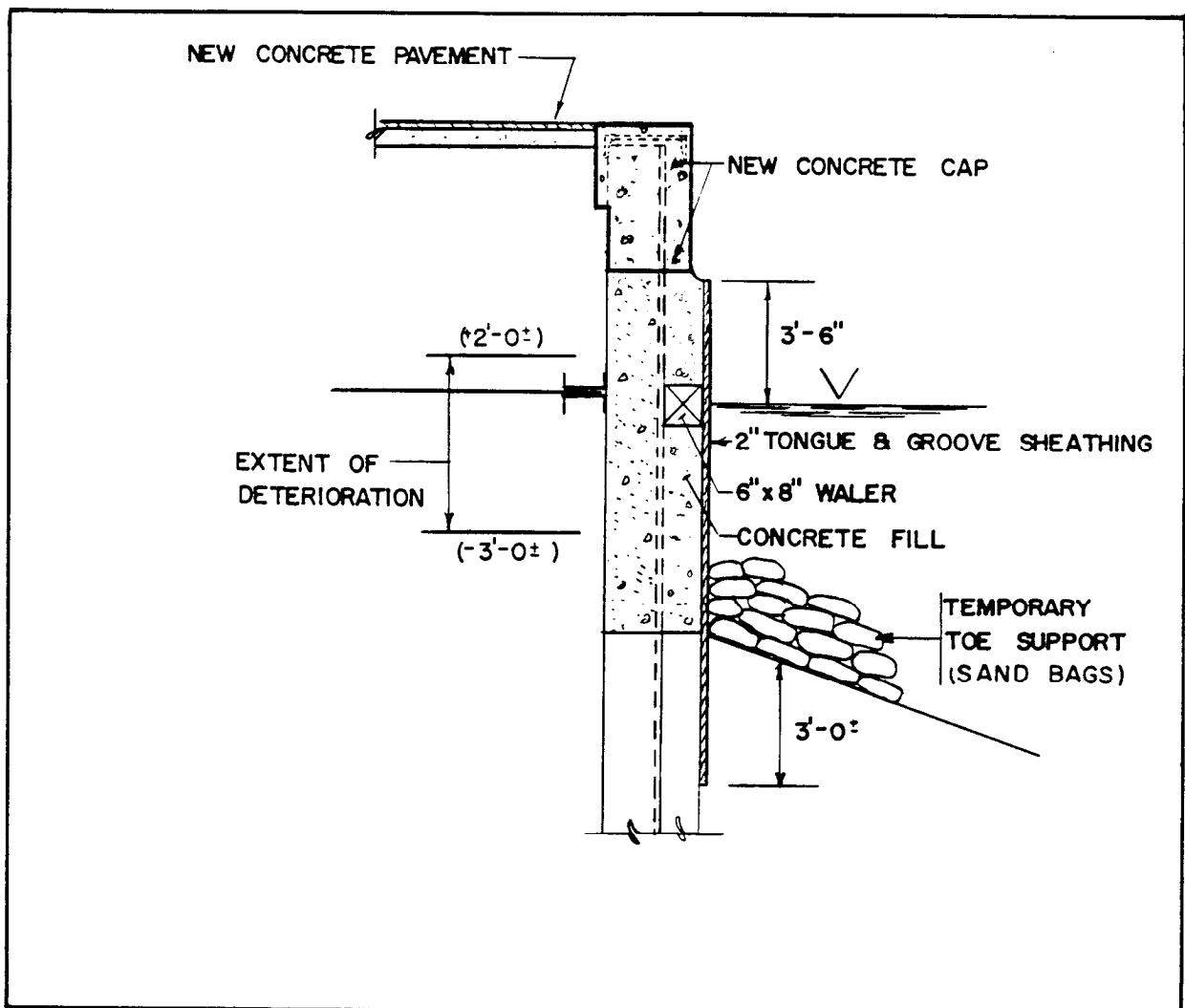


Figure 39  
Case History No. 2--Repair Details

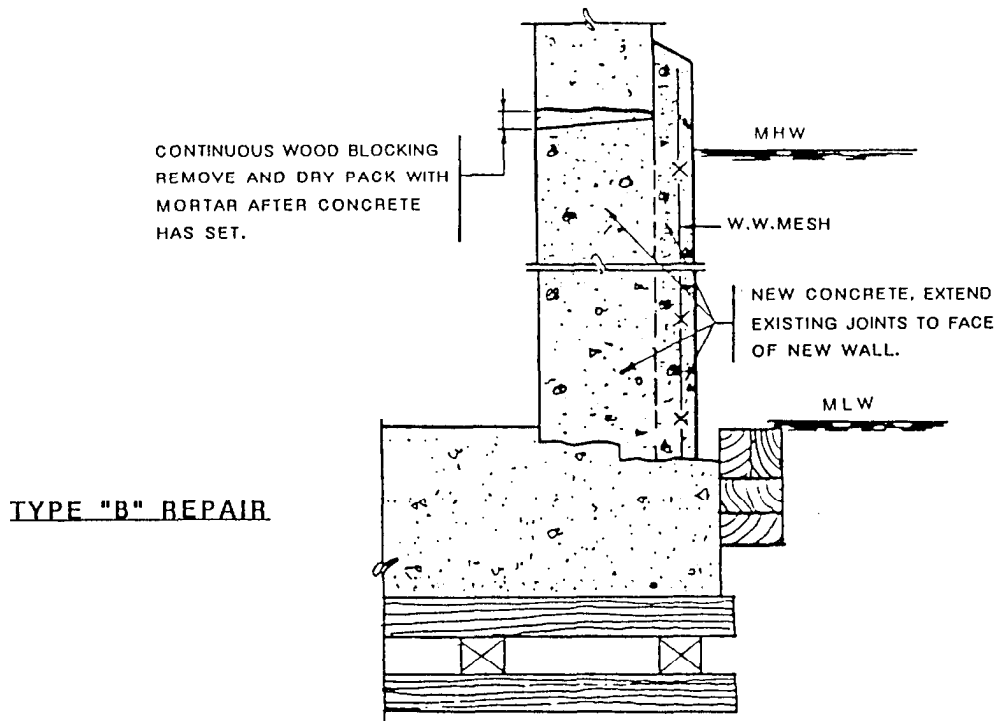
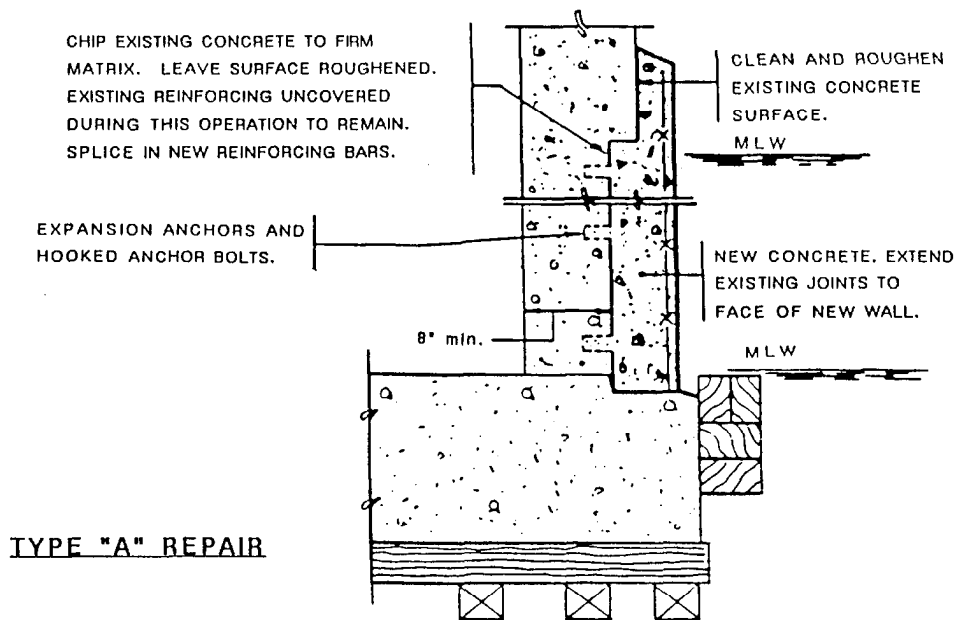


Figure 40  
Case History No. 3--Repair Details

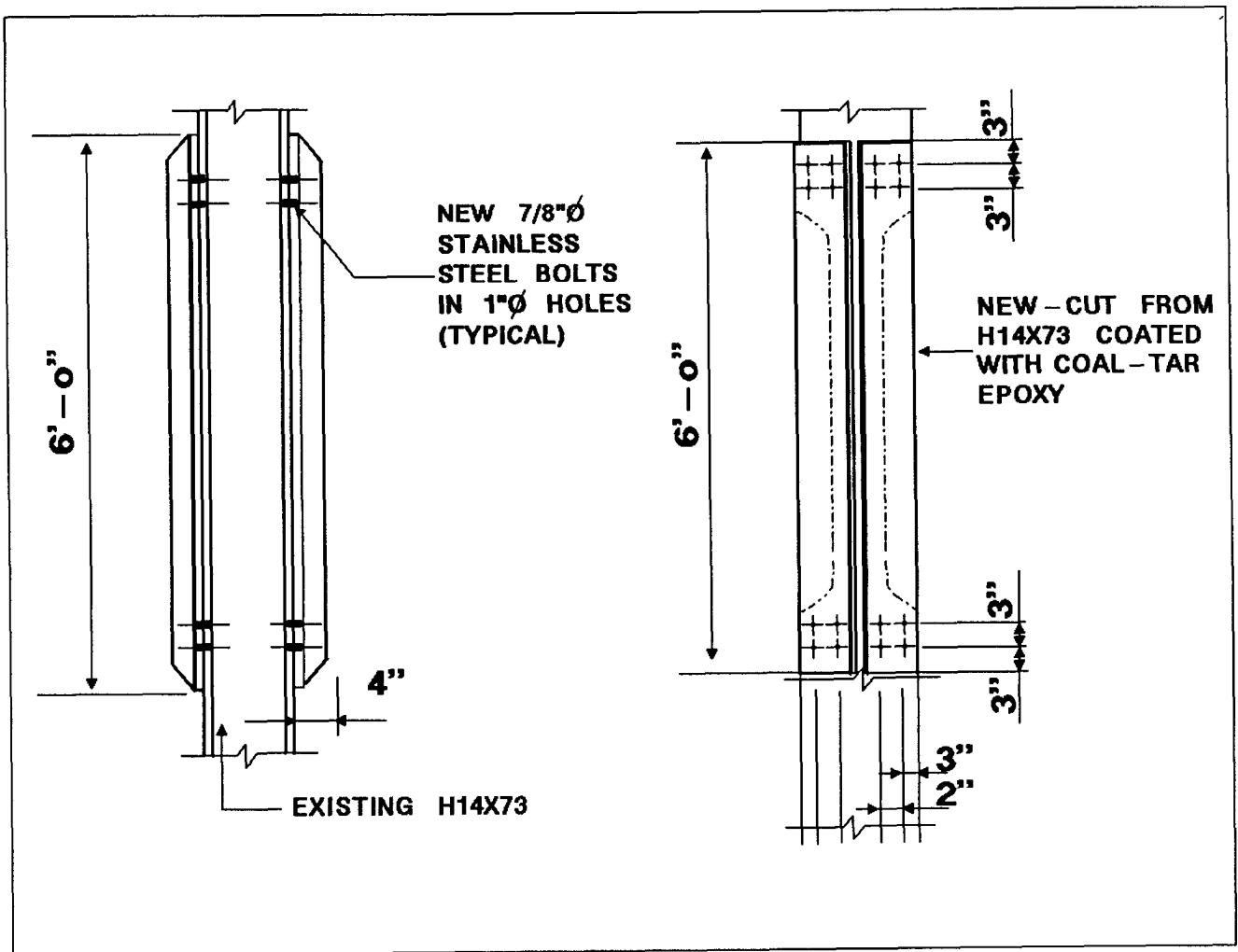


Figure 41  
Case History No. 4--Repair Details

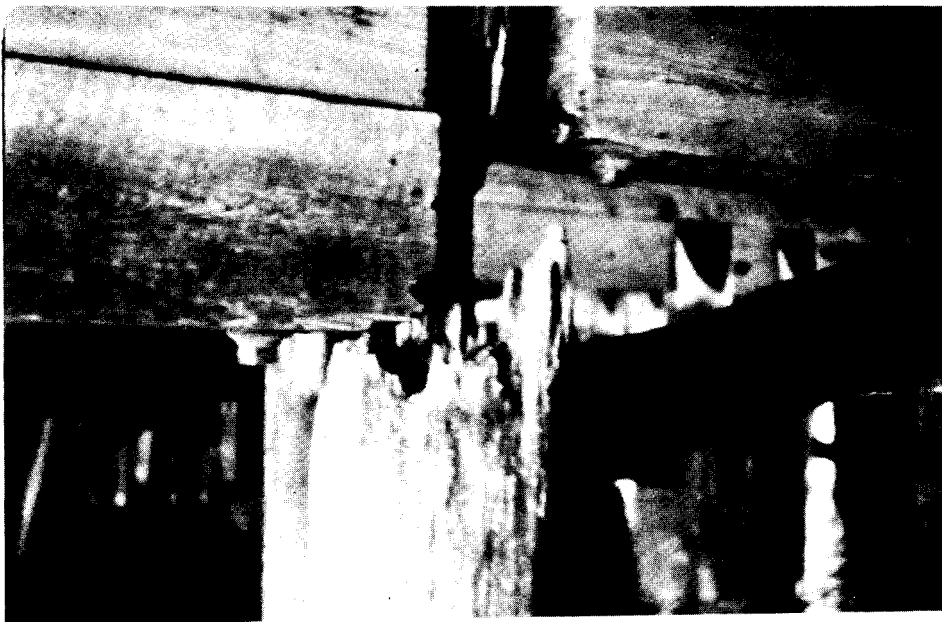
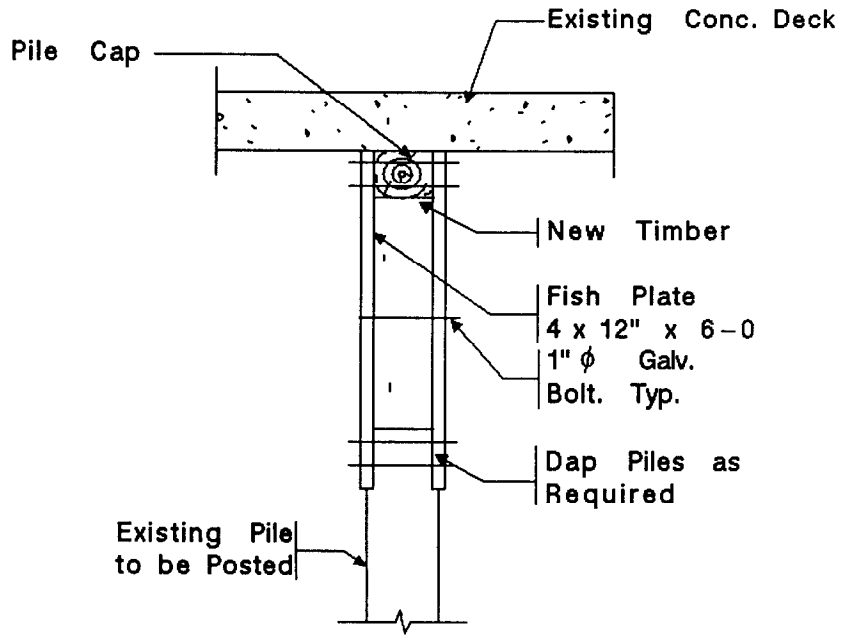
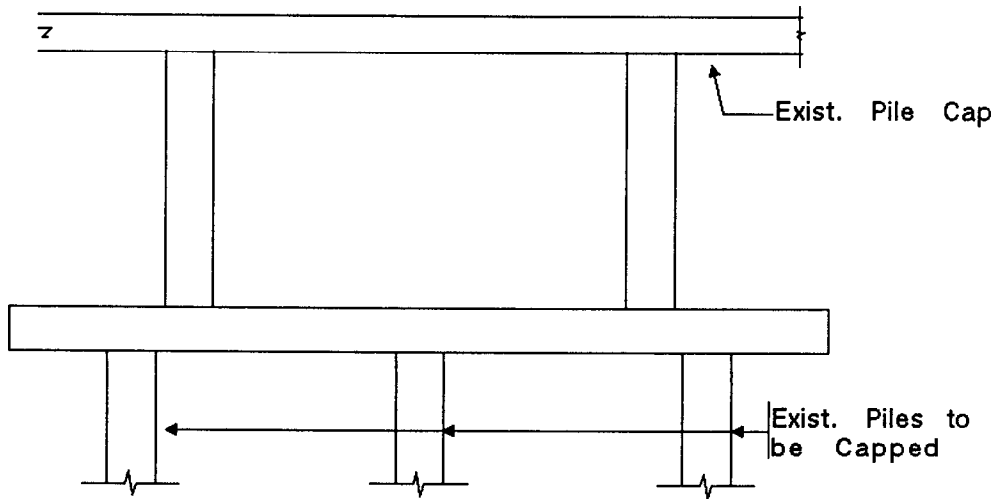


Figure 42  
Case History No. 5--Deterioration of Timber Pile



**PILE POSTING DETAIL**



**BENCH CAP**

Figure 43  
Case History No. 5--Repair Details

### 7.3.6 Case History No. 6--Sulfate Corrosion of Concrete Piles

7.3.6.1 The Problem. Deterioration of precast concrete piles due to sulfate attack (see Figure 24). Piles were made using Type III (high early strength) cement. Attack extended to mudline.

7.3.6.2 The Solution. Jacket with concrete (see Figure 44).

### 7.3.7 Case History No. 7--Biological Attack

7.3.7.1 The Problem. Attack by Limnoria lignorum and Pholadidea of creosoted timbers and piles in Bermuda. See Figure 45 for cross section of structure.

7.3.7.2 The Solution. Piles were wrapped with plastic sheeting. Design was checked and upon determining that it was structurally feasible, pile bracing was raised above water (see Figure 46).

### 7.3.8 Case History No. 8--Corrosion of Steel H-Piling (Untreated)

7.3.8.1 The Problem. Corrosion of steel H-piling. See Figure 12 for a typical cross section of the pier.

7.3.8.2 The Solution. Do nothing! (judicious neglect). Figure 5 shows profile of residual thickness of metal. Calculations show enough column capacity remaining to provide substantial service life before critical conditions would develop. Economic analysis in accordance with NAVFAC P-442, Economic Analysis Handbook, showed overwhelming advantage to deferring repair. With high discounting rates (8 percent or more), this is the usual conclusion.

### 7.3.9 Case History No. 9--Corrosion of Steel Sheet Piling

7.3.9.1 The Problem. Corrosion of steel sheet piling of cell structure and of steel framing supporting the fender system. Figure 4 shows corrosion profile for steel sheet piling.

7.3.9.2 The Solution. Steel framing was reinforced by plating as shown in Figure 47. Only webs had substantially thinned. Flanges were free-draining and had lost negligible section. Jacket cells in active corrosion zone using detail shown in Figure 48. Active corrosion zone was of limited height -- at or near mean low water.

### 7.3.10 Case History No. 10--Corrosion of Steel Bracing

7.3.10.1 The Problem. Corrosion of steel bracing for piling.

7.3.10.2 The Solution. The braces were removed and were not replaced. Careful analysis indicated that braces were not necessary adjunct if restraint of trestle structure due to limited movement which could develop before deck butted against cells and approach abutment was considered.

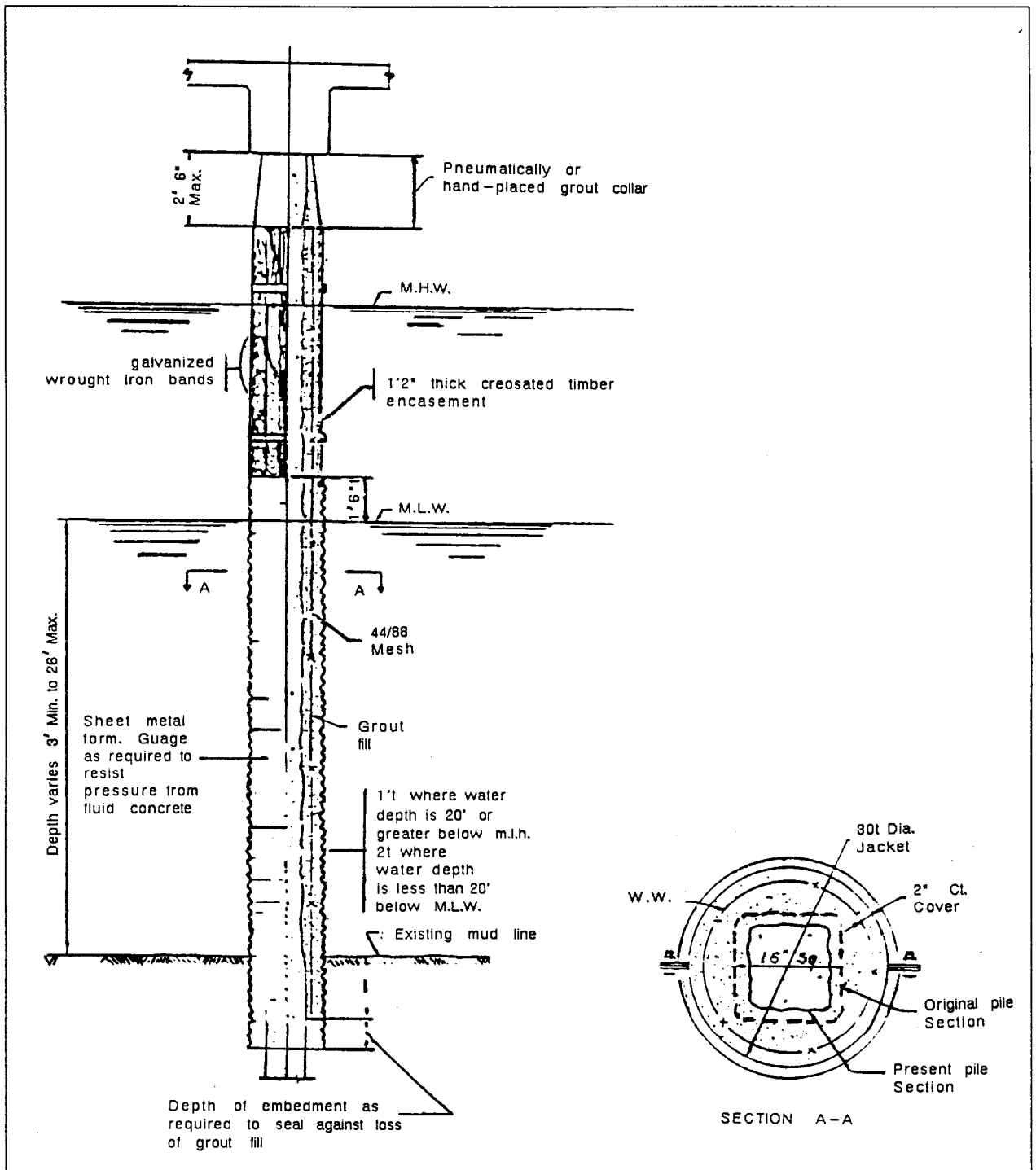


Figure 44  
Case History No. 6

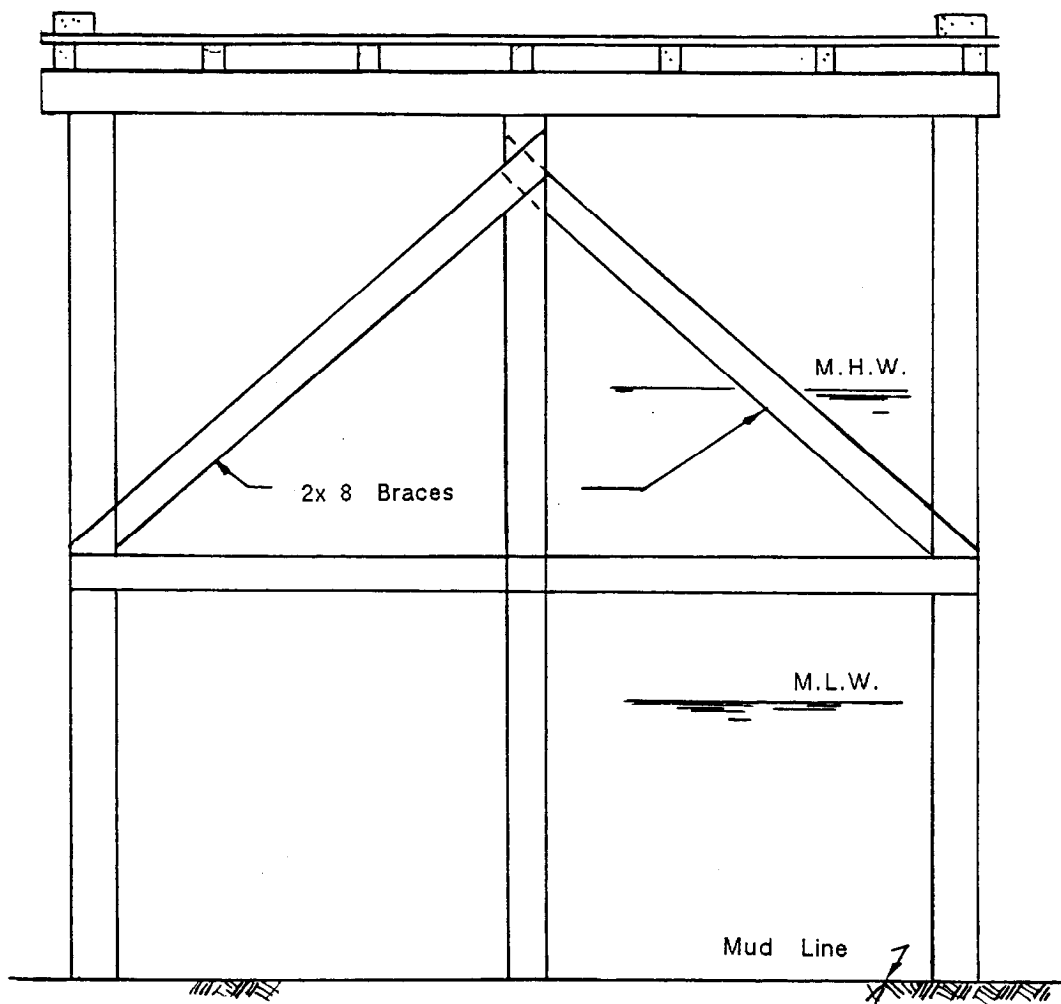


Figure 45  
Case History No. 7 - Cross Section of Existing Pier

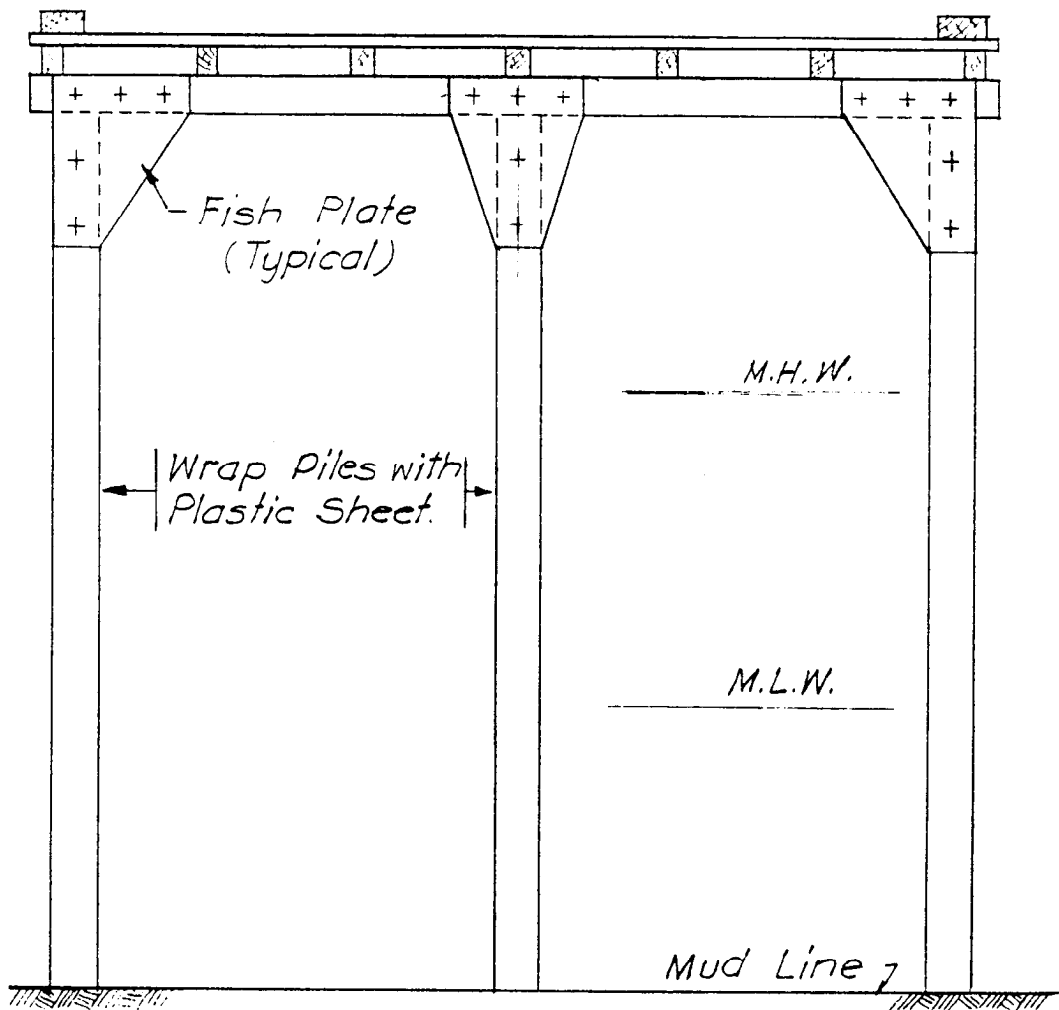


Figure 46  
Case History No. 7--Repair Details

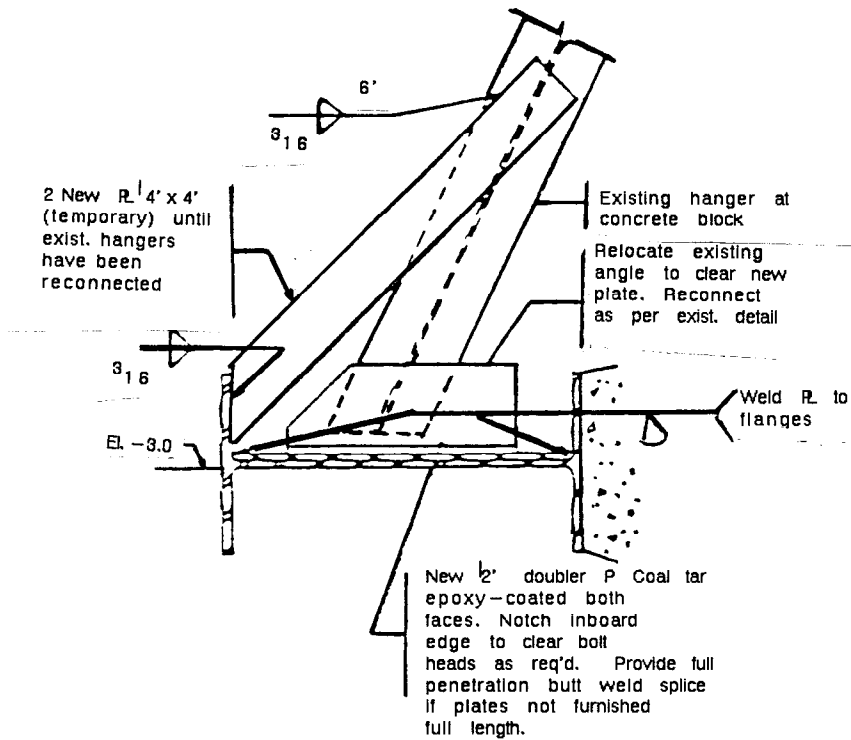


Figure 47  
Case History No. 9 - Repair Details, Steel Framing

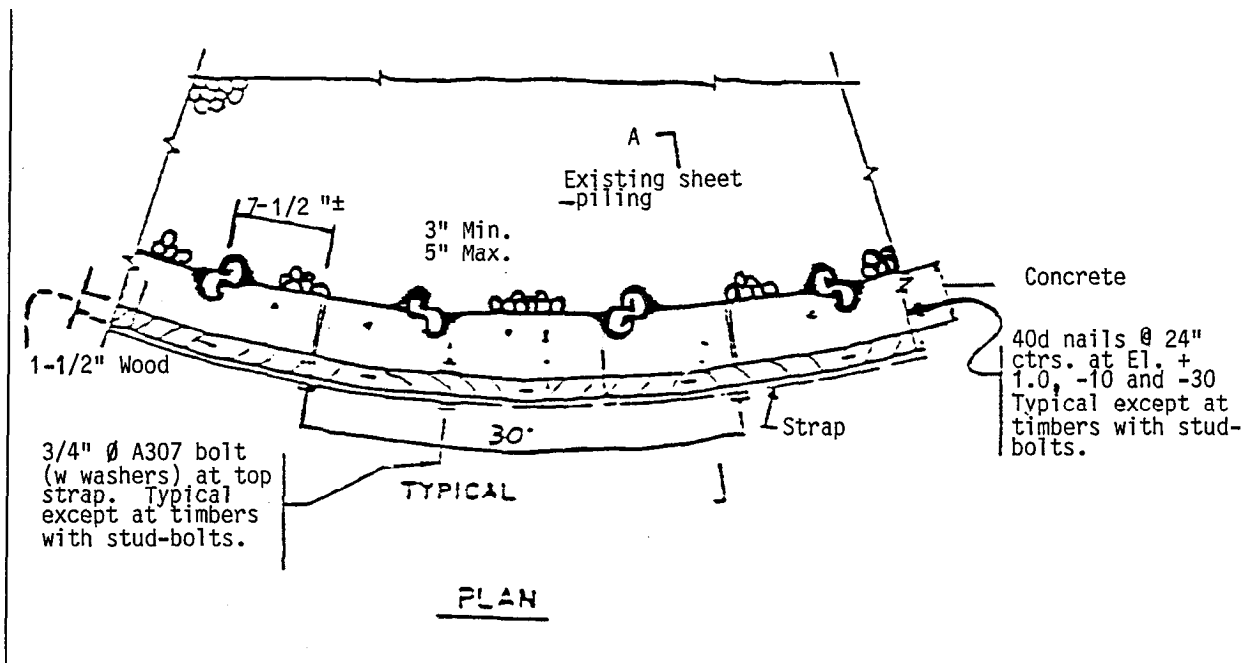


Figure 48  
Case History No. 9

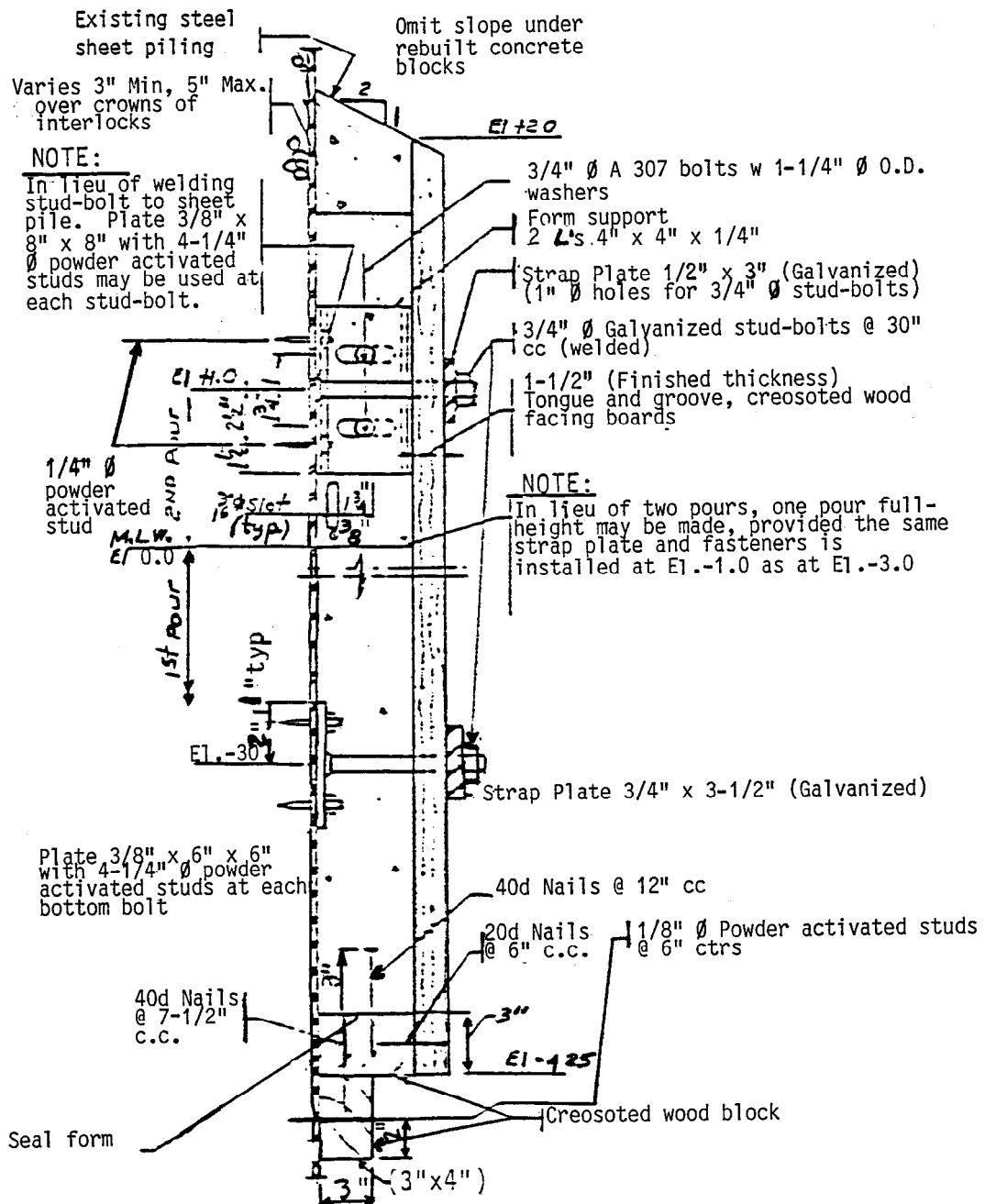


Figure 48  
Case History No. 9

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NAVFACENGCOM Design Manuals (DM) and Guide Specifications (NFGS)

Department of Defense activities may obtain copies of Design Manuals, P-Publications, and Definitive Drawings from the Commanding Officer, Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120. Telephone: Autovon (DoD only) 442-3321; Commercial: (215) 697-3321. Department of Defense activities must use the Military Standard Requisitioning and Issue Procedure (MILSTRIP), using the stock control number obtained from NAVSUP Publication 2002.

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DM-2.5	Timber Structures
MIL-HDBK-1025/2	Dockside Utilities for Ship Service
DM-25.4	Seawalls, Bulkheads, and Quaywalls
NFGS-02361	Round Timber Piles
NFGS-02367	Prestressed Concrete Piling
NFGS-02368	Rolled Steel Section Piles
NFGS-02363	Cast-in-Place Concrete Piles, Steel Casing
NFGS-02371	Auger Placed Grout Piles
FCGS-02360.1	Round Timber - Concrete Composite Pile

## REFERENCES

ACI Standard 318, Building Code Requirements for Reinforced Concrete, available from American Concrete Institute, Box 19150, Redford Station, Detroit, MI 48219.

American Society for Testing and Materials (ASTM), 1916 Race Street, Philadelphia, PA 19103.

ASTM A6	Standard Specification for General Requirement for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use
ASTM A36	Specification for Structural Steel
ASTM A242	Specification for High-Strength, Low-Alloy Structural Steel
ASTM A252	Welded and Seamless Steel Pipe Piles
ASTM A588	Specification for High-Strength, Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick
ASTM A690	The Specification for High-Strength, Low-Alloy Steel H-Piles and Sheet Piling for Use in Marine Environments
ASTM A709	Specification for Structural Steel for Bridges
ASTM C42	Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
ASTM D25	Specification for Round Timber Piles
ASTM D245	Standard Methods for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber

AWPB Standards, American Wood Preservers Bureau, P.O. Box 6085, 2772 S. Randolph Street, Arlington, VA 22206.

MP-1	Standard for Dual Treatment of Marine Piling Pressure Treated with Water-Borne Preservatives and Creosote for Use in Marine Waters
MP-2	Standard for Marine Piling Pressure Treated with Creosote for Use in Marine Waters
MP-4	Standard for Marine Piling Pressure Treated with Water-Borne Preservatives for Use in Marine Waters
MLP	Standard for Softwood Lumber, Timber and Plywood Pressure Treated for Marine (Saltwater) Exposure

National Fire Protection Association (NFPA), Batterymarch Park, Quincy, MA 02269

NFPA 87 Construction and Protection of Piers and Wharves

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DM-2.04	Concrete Structures
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DM-7.03	Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction
DM-26.1	Harbors
DM-26.06	Mooring Design Physical and Empirical Data
MIL-HDBK-1002/1	Structural Engineering: General Requirements
MIL-HDBK-1004/10	Cathodic Protection
MIL-HDBK-1002/3	Steel Structures
MIL-HDBK-1008A	Fire Protection for Facilities Engineering, Design, and Construction
MIL-HDBK-1025/1	Piers and Wharves
OPNAVINST 5510.45B	U.S. Naval Physical Security Manual
P-442	Economic Analysis Handbook
P-272	Definitive Designs for Naval Shore Facilities

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